REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Cannon Beach Cache Site Police Project
Cannon Beach, Oregon

Geotech Solutions Inc.

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As authorized, herein we present our report of geotechnical engineering services for the proposed Cache Site Police Facility in Cannon Beach, Oregon. A roughly 5,300 square foot single story wood framed structure is planned, with associated pavements and utilities. We previously provided consultation on this site for storage projects in 2013. Recently we were provided preliminary geotechnical and geological reports by others that included the "Southwind" project abutting this site, as well as instrumentation monitoring of ground water and subsurface movement, and have used this as background for our work. From that previous work, seismic landslide induced deformations were expected, and geotechnical analyses and recommendations were needed for building design performance by others. That performance is expected to include having the building life safe and functional after a CSZ design level earthquake, albeit with some level of damage. The overall purpose of our work was to complete project specific site explorations and analyses to provide recommendations for building design. As an essential facility, our work also included site specific evaluation of seismic hazards including seismic stability for the proposed building support design compatible with the complexity of the project. Specifically, our scope included the following:

- Provide principal level geotechnical project management including a site reconnaissance, review of provided information, client communications, and review of analyses, reports, and standard format invoicing.
- Review previous reports, geologic maps, and vicinity geotechnical information as indicators of subsurface conditions.
- Complete a site reconnaissance and mark the exploration locations.
- Complete one-call utility locates and subcontract a private locator. Utilities that cannot be located (i.e., plastic, non-ferric, no tracer wire, etc.) are the responsibility of the owner and may be damaged if not marked. Damage to these and exploration locations, and surface repair of any kind, other than backfilling and any asphalt patching of explorations, is not a part of this scope.
- Explore subsurface conditions by advancing two mud rotary drilled borings to depths of up to 100 feet or refusal or 20 feet into basalt, and 3 test pits with an excavator to depths of up to 10 feet or refusal.
- Maintain a detailed log of the explorations and obtain samples at intervals and make observations for evidence of ground water.
- Complete laboratory testing to aid in soil classification.
- Evaluate site specific seismic hazards: including tsunami, fault rupture, and complete detailed liquefaction analyses of site soils, and estimate liquefaction induced deformations and provide qualitative means to reduce deformations as needed.
- Complete static and dynamic slope stability analyses in a 2D cross section of the site from the borings and previous adjacent work, including the overall site and means to reduce hazards.
- If feasible, provide recommendations for new shallow reinforced mat foundation or grade beam

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support, including possible reinforced subgrade, and criteria and forces for resistance to lateral loads and movement, as well as settlement from static loads, site preparation and base rock, and foundation drainage.

- Provide shear pile analyses for one pile type to reduce deformations, including pile size, type and spacing, estimated embedment and possible use outside the building footprint.
- Provide recommendations for site grading, including earthwork vertical extent limitations regarding stability, wet season grading criteria, surface soil stabilization for pavements, and utility backfill materials and compaction.
- Provide recommendations for site pavement thicknesses and materials.
- Provide a PE/GE stamped written report summarizing the results of our geotechnical evaluation.

SITE OBSERVATIONS AND CONDITIONS

Surface Conditions

The site consists of gently- to moderately sloping terrain, with extensive past filling and earthwork associated with the gravel loop road and levelling/site grading in the proposed building area. A buried, culverted drainage is present under the fills presumed running east-west across the center-north of the parcel with the culvert outlet near the north drive at the Highway. A storage building and containers are present in the east central portion of the parcel northwest of the drainage ravine and culvert inlet. The drainage ravine may have been created from filling west of it, and now routes to the west-southwest toward the Tolovana Mainline Road intersection. No significant foundation cracking or distortion of the storage building was noted during our site work. Mike McEwan of McEwan Excavating recalled historical filling and described mixed fill materials with predominantly organic soils and topsoil fill to the north side of the loop road, and inorganic soils within the loop including some concrete rubble.

Aerial photos of the site were obtained from the Corps of Engineers, City of Cannon Beach archives, and Google Earth historical photos. Photos were reviewed for grading and surface changes to the degree possible by the generally low resolution and are attached to this report. Features included the following:

1939 - The site is forested with what appear to be mature conifers, and the central drainage is unfilled. Highway 101 is not present, and Hemlock Street is present but appears unpaved.

1962 - The site area has been cleared and the central drainage is present and appears to be culverted under Highway 101 which is also present. The Loop road is present further north than the 2013 alignment.

1977 - Some regrowth of brush is present on the site, with additional filling near Highway 101 and over the west end of the ravine.

1994 - Extensive additional fill is evident over the ravine area, and the ravine is not evident in the cleared area. The northernmost loop road is overgrown with brush and a new northern loop road is evident further south. Possible fill tiers are present.

2001 - The site fills have low vegetation present over them, with some active filling of the lower south loop road near the highway.

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2012 - The storage shed is present, and new fills are evident in the center of the loop road.

Subsurface Conditions

Geologic maps indicate the site is within marine terrace deposits of silt and clay (City of Cannon Beach Geological mapping, DOGAMI Bulletin 74, DOGAMI ODGC-7). These maps indicate this unit is underlain at depth by sedimentary rock of the Astoria Formation with Columbia River basalt present to the southeast. Bulletin 74 indicates that the Astoria Formation mapped to the east northeast is part of landslide terrain and this parcel appears to be within that or at the southern margin. The city mapping does not indicate that the site is in active landslide terrain, nor does Oregon SLIDO. Inclinometer readings over a period of 3 years in B-3 by others just east of the site boundary indicated no movement from 2020-2023, and the Southwind preliminary report states that the slide is not considered active. Personal communications and mapping with/by Tom Horning of Horning Geosciences (excerpt attached) from work on the Southwind site indicate the site as marine terrace south of an incised drainage, with bedrock Astoria Formation contact generally at elevations higher than 120 feet east and south of the site, and outcropping basalt in ridges further southeast. The isolated "mound" feature above the site on the City access road may be a slide feature as a test pit by Horning in that area described conditions as possible basaltic colluvium. We observed a basalt outcrop at elevations below roughly 200 feet east-southeast of the site east of the incised drainage that appeared massive and may represent a slide block or a thick intrusive body/sill.

To evaluate site specific soil conditions, we advancing three test pits to depths of up to 15 feet with an excavator, and 2 borings to depths of up to 100 feet at the approximate locations shown on the attached ***Site Plan***. A previous boring by others in work for the Southwind site was advanced to a depth of 150 feet at the "B-3 by others" location just off site to the east as shown on the ***Site Plan,*** and a well log from the T-Mobile site uphill to the southeast that encountered siltstone was also reviewed. 5 test pits were also reviewed from our 2013 report on the site for storage.

The subsurface consisted of several units of soil and rock. These generally included from the surface down; fill, younger terrace landslide deposits, older non-landslide terrace deposits, siltstone (where present), and basalt. These units are described in the following sections, with strength and other parameters of each unit used in our stability analyses summarized in the attached stability sections.

Fill - Fill content varied widely in both the current and previous 2013 test pits. Materials in previous test pits north of the loop road included very soft organic silt with debris to depths of 7 to 9 feet overlying a 2-foot-thick layer of crushed rock fill in one test pit, with a layer of buried original rooty organic topsoil beneath both. Fill in our current explorations extended to depths of roughly 10-11 feet, and consisted of variable fine sand, silt, gravel, occasional concrete, asphalt, and boulders and scattered trace organics. Blow counts (N_{85} autohammer) in the fill ranged from 5-24, with moisture contents of 13-37% in current test pits, and up to 94% where organic in previous northern test pits. Minor to moderate caving was common the test pits. Despite the medium stiff or better condition, this fill is inconsistent and undocumented and not have the reliable properties of structural fill. Previous explorations north of the loop road encountered that fill as including organics, and Mike McEwan stated after topsoil loads were routed to fill in that location.

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Upper Terrace Silt/Ancient Slide Deposit - Beneath the fill and topsoil, soils consisted of very soft to stiff generally gray silt with variable sand and clay content and variable wood debris that extended to depths of 43 to 48 feet in our borings, and 33 feet in the B-3 boring by others to the east. The blow counts ranged from 0-22 with most below 10 and many of 5 or less, and generally softer more variable structure in the lower 10-20 feet. Plasticity ranged from non-plastic to moderately plastic where clay content was higher. Moisture contents ranged from 29-197%, the higher readings correlating to organics. Previous strength testing in the soil by others and in our experience indicate a static phi angle of 10-15 degrees in the softer zones of this unit. The strength is considered higher in cyclic loading related to the number of loading cycles which are high for CSZ interface earthquakes. Carbon dating by others of the wood debris in the upper terrace indicated the wood was growing roughly 20-40 thousand years ago. Much of the wood observed in this unit was still relatively fresh and undecayed in our samples.

Lower Terrace - This unit was present under the upper terrace in both of our borings and extended to a depth of 58 feet in B-1 to the east and 82 feet in B-1 to the west. The unit includes an undisturbed sedimentary structure of silt to sandy silt with variable fine organic content and is inferred as not landslide deposited. The unit was generally stiff with blow counts of 8-15 with two exceptions. The upper few feet in B-1 was very soft with trace fine sand and organics (which could represent old topsoil). The lower 7 feet of this unit in B-2 consisted of very dense fine poorly graded sand with blow counts of 86 to 50/5", consistent with our vicinity downslope borings and inferred as wave densified (and/or seismically densified) ancient beach sand common lower in this unit.

Siltstone - Siltstone was encountered at a depth of 48 feet in B-1. The top roughly 5 feet of the unit was severely weathered into silt with siltstone clasts retrieved as gravel in size with a blow count of 6. Below this extending to a depth of 58 feet the siltstone was soft to moderately hard rock and little weathered, with a blow count of 53. This unit was also encountered beneath the landslide terrace in B-3 upslope and extended to the 150-foot depth explored in that boring. Siltstone was not encountered in our boring B-2.

Basalt – Hard, little weathered, fractured, dark gray to black basalt was encountered at depths of 58 to 82 feet from east to west, in B-1 and B-2, respectively, but was not encountered in B-3 by others. Attempted coring of the basalt was very difficult due to fracturing with little retrieved, and a tricone bit was then used with the CME 75 drill rig with advance rates of 6 to 8 feet an hour. Basalt was observed outcropping (or in a large slide block) at about 200 feet in elevation east-northeast of the site and is mapped in the hillside to the southeast. The basalt is generally intrusive and displaces the siltstone and is interpreted as massive below the building site or a very thick sill or body and was not present in B-3 by others nearby. This basalt would preclude deeper slide surfaces perhaps corroborated by overlying older marine terrace deposits remaining undisturbed.

Groundwater - Wet soil conditions were noted at depths near 20 feet in our borings and were not encountered at depths of 15 feet in our test pits, done near the end of the dry season. Instrumentation and monitoring in B-3 by others showed wet season groundwater levels near 15 feet in depth in an inferred perched condition. The B-3 boring included instrumentation in sealed zones at depths of 100 to 150 feet but did not observe excess confined pressures.

Slope Stability Analyses
As discussed previously, the site area is considered to be blanketed in ancient landslide deposits that are not active but can be destabilized in earthquake motions. To evaluate stability we used several sources of information to develop stability models. This includes City GIS 2-ft topographic information, site reconnaissance of outcropping units, geological mapping, site explorations, and testing of encountered units in both our site sampling and experience in the vicinity and on the Southwind site by others. Based on our local experience in these units we also considered the likely presence of a weak shear zone in the lower portions of the landslide terrace unit. Morgenstern-Price limit equilibrium methods were used, and sensitivity analyses were conducted on each of these parameters along with ground water levels to refine the inputs and evaluate their impact.

From the preceding information and approach we used the stability software SLIDE2 and embedded seismic deformation program SLAMMER's Newmark analyses to evaluate the probable stability of each model, seismic yield accelerations, and expected seismic deformations. Two primary 2D profiles were evaluated based on the most probable instability cross sections, as shown on the attached Sections. The more east-west line in Section A-A was found to have the lower stability, with a static factor of safety of 2.6 and a yield acceleration of 0.26g. As the site is near the margin of more stable conditions to the south and southeast, these are likely somewhat conservative if 3D influences are considered. To estimate deformations during the design level 0.73 accelerations (for a magnitude 9.0 CSZ interface quake) we used the SLAMMER Newmark analyses in both scaled earthquake time histories and empirical estimates (Jibson'07, Saygili Rathje '08) as well as independent empirical subduction zone modeling estimates (Macedo '17). For the most applicable scenarios, this resulted in estimated site deformations along A-A ranging from 3 to 8 inches laterally parallel to the shear surface (inclined slightly down of horizontal to the west). Typical estimates of vertical deformation are half the lateral, which would be about 2 to 4 inches. Half of that in differential settlement would also be typical, at 1-2 inches.

Southeaster Hillside Stability - No significant slumps or indications of large-scale instability were noted in our reconnaissance of the southeast hillside and review of LIDAR imagery. The hillside is generally sloped at 1.8H:1V to 2.5H:1V. Old logging road/skid road cuts generally have localized raveling exacerbated by game trails, but no significant or fresh slumping was observed. Some of the large spruce trees show slight overcorrected growth, likely due to surface soil creep. This slope has an age subjected to many CSZ interface earthquakes and does not show features of past global instability. It is possible that shallow or "veneer" slides could occur in wet season seismic conditions. Thin flow slide runout is possible but unlikely to impact the location of the building footprint due to site topography and typical inviscid behavior following site topography. To reduce this risk and divert possible flows, the eastern site berm could be enhanced to route flow toward the southwest entrance drive away from the building. If flow materials reach the lower drive area, such materials can typically be excavated/removed with conventional equipment.

Stabilization – If needed, one option to reduce deformations may be shear piles that could double for building support. This type of pile essentially increases the resisting forces along the shear surface of the slide and can also carry vertical building loads. Our stability analyses indicated that 200 kips in shear per pile, with piles at 8 ft centers under the building pad, would increase the yield acceleration to 0.35g and lateral deflection estimates to about 3 inches. This has been done on other sites with a drilled reinforced concrete piles, but typically in a scenario where the slide zone overlies a much stronger more

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rigid unit. As expected, due to the depth of the shear surface of 43 to 48 feet and the thickness of the underlying lower marine terrace over basalt, bending moments for piles at the interface were very high. For example, for a 200-kip pile shear capacity a 4-ft diameter drilled reinforced concrete pile with 14 #14 bars properly seated 10 feet into basalt (a total depth of 68 feet in B-1 and 92 feet in B-2) would develop a plastic hinge at only roughly 4 inches of movement on the shear surface. In conclusion this method would only reduce total estimated deformation from 8 to 4 inches, and at a very high pile cost.

Other methods of increasing resistance across the shear zone could be used for the stabilization at an equivalent shear load across the building, such as jet grouting or ground anchors. These likely have an even greater cost than the preceding pile approach. Reducing groundwater levels was considered but is likely impractical due to the low permeability of the terrace soils and slide dimensions. Loading and unloading of the site area was also considered impractical due to the small size of the site to the overall slide, as well as possible downslope localized stability impacts.

CONCLUSIONS AND RECOMMENDATIONS

The preceding estimated deformations are generally moderate for low-rise structures in this situation but would likely cause structural damage for conventional spread and continuous footing construction. In conference with the structural engineer at CIDA, we discussed the preceding shear pile to increase performance, albeit at a high cost and moderate gain (3-4 inches of total deflection versus 8 inches). The moderate differential seismic slide deformations and light building may also allow for a reinforced mat or grade beam system, supported by lighter piles to reduce settlement risk for gravity (non-landslide) loads. The mat/grade beams would serve to reduce differential movement of the structure in an earthquake condition, and the piles would be used to reduce static settlement from the uncontrolled fill and underlying upper terrace deposit in non-earthquake conditions. The structural engineer may be able to design this system to reduce building damage to an acceptable performance level, and geotechnical parameters for design of such a system are included in the following ***Foundations*** section.

The deformations in an earthquake may damage utilities, especially less ductile conduit or conduit with little tension capacity at the joints. The preceding differential movement in the ***Stability Analyses*** section can be used to evaluate utility performance, and consideration of flexible connections, alignment, materials, and allowance for deformation should be made. It would likely be prudent to include emergency power and communication systems contiguous with the reinforced mat or grade beam system of the building to reduce risk to emergency systems.

It should be noted that the total lateral slide deformation estimated at 8 inches is only an estimate based on the described analyses. More or less deformation may occur as the analyses is complex with many variables. Based on the references used, the deformation estimate presented was the highest of those calculated, and for the subduction zone empirical model estimate (Macedo '17) generally has an 84% level of not being exceeded for the motions used.

Seismic Design

The response of the project site soil profile in proposed building areas is consistent with site class D. Ground motion parameters for this site at a code level of 2% chance of being exceeded in 50 years are included in the attached ASCE 7-16 hazard tool output and include a PGA of 0.73g. In addition to these parameters the project design team should understand that repeated cycles of horizontal ground

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accelerations from the relatively near field Cascadia Subduction Zone (CSZ) interface earthquakes are expected to be in the 0.3-0.5g range, with duration of strong motion of several minutes. Refer to the ***Seismic Hazard Investigation*** herein for more detail on the level of seismic hazards.

Foundations

Based on our analyses and discussions with the structural engineer, in our opinion the most cost effective foundation system for building support to a functional performance may be a reinforced mat or grade beam system with a structural slab. To reduce settlement from static/gravity loading, helical piers could be used. The following sections provide parameters for this system.

Mat or Grade Beams - A reinforced mat foundation or grade beam system can be designed for tensional forces during lateral movement that would be acting to pull the mat or beam system apart. These forces would consist of frictional forces on the north and south sides and the base of all grade beams or mats. An ultimate base friction coefficient of 0.39 should be used on the base (this assumes the existing fill is under the grade beams). A side friction coefficient of 0.22 can be applied to the sides with a normal force from the lateral pressure of a 30 pcf equivalent fluid. As helical piers are expected to fail laterally given their low moment resistance, the grade beams should also be sized for a width that accommodates an allowable bearing pressure of 1,500 psf for post-earthquake movement support. This pressure is not expected to result in more than 2 inches of settlement post-earthquake from the gravity loads, and the strength of the grade beams would likely allow for levelling pier applications if needed.

A minimum of 12 inches of clean, angular crushed rock with no more than 5% passing a #200 sieve is recommended for base rock under slabs or a mat. This can be substituted for the recommended working pad in the ***Earthwork*** section of this report only if it remains clean and uncontaminated with fines. Prior to slab placement the rock will need to pass a wheel roll with a fully loaded truck or meet 92% compaction relative to ASTM D-1557, or approval via probing by the geotechnical engineer. In addition, any areas contaminated with fines must be removed and replaced with clean rock. If the base rock is saturated or trapping water, this water must be removed prior to slab placement. Two inches of crushed rock is recommended under grade beams to keep an undisturbed condition.

We recommend slabs be designed to free span between grade beams. We recommend a vapor barrier be used under the slab or mat. Typically, a reinforced product or thicker product (such as a 10-15 mil STEGO wrap) can be used. Experienced contractors using special concrete mix design and placement have been successful placing concrete directly over the vapor barrier which overlies the rock. This avoids the issue of water trapped in the rock between the slab and vapor barrier, which otherwise requires removal. In either case, slab moisture must be tested/monitored until it meets floor covering manufacturer's recommendations.

Drilled Shafts for Slide Shear Improvement

To reduce seismic deformations to roughly 3 inches laterally and less than 2 inches vertically, the preceding stabilization piles (drilled shaft 4 feet in diameter with 14 #14 bars) could be used and support over 200 kips per pile vertically, and 15 kips per pile laterally in static loads, if embedded at least 6 feet into basalt, or 10 feet into siltstone. Piles would need to be installed at 8-foot centers north to south, and 12 feet east to west to accumulate enough total shear to reduce deformations to the preceding 3 inches laterally. Downdrag loads from organic decay are included in the preceding but are not expected

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to be large as primary organics were relatively undecayed ancient debris not expected to induce enough settlement for full mobilization. Pile sequencing would likely require drilling of every other pile during construction sequencing to reduce potential caving or grout loss, and casing is expected to be required above the siltstone or basalt. The cost of the preceding piles may not justify the modest deformation reduction if the alternative grade beam or mat system can be suitably designed.

Helical Pier Foundations

Provided the preceding mat or grade beam foundation system is used, helical piers can be designed to support static/gravity loads and reduce settlement from underlying uncontrolled fill and the soft portions of the upper terrace soils. Installation of helical piers may not be feasible to the required depths, and reaching these depths must be proven with the use of indicator piers. Occasional boulders and debris were present in the upper fill. For moderate loading up to 40 kips, pier embedment of at least 10 feet into the underlying lower stiff terrace and correlated capacity torques can be used. Based on our explorations, the top of the lower terrace unit ranged from 43 to 48 feet below the ground surface, so resulting pier lengths would be 53 to 58 feet below the existing ground surface, although this is expected to vary widely. A tensional load test is required prior to production pile installation, tested at 50% intervals to 200% of design pullout capacity with creep measurements at the design load.

Capacities listed herein may be limited by the structural capacity of the pile and must be evaluated by a structural engineer. Piers must be spaced a minimum of 3 pile diameters apart. Closer spacing will result in reduction in pier capacity and we must be consulted. Fills greater than three feet above existing grades in the building pad will induce down-drag on the piles and are not recommended unless they are installed at least 6 months prior to construction, are adequately monitored for settlement with at least 3 settlement plates, and if such monitoring indicates settlement is complete prior to pile installation. Settlement could take longer.

Piers in a fixed condition in grade beams are recommended. Due to the risk of long-term settlement we recommend floors be designed as structural to free span between grade beams or be directly pile supported. Interior unsupported slabs-on-grade are not recommended.

We recommend vertical piers with the following allowable capacities be used for design, with a minimum pier spacing (vertical and horizontal) of three helix diameters. Resistance to non-seismic lateral loading of 1.5 kips per pile is allowed for vertical piles, and piles battered up to 30 degrees from vertical can be designed to the horizontal vector of the preceding loads in the horizontal direction of downward batter, and 90% of that in the opposite horizontal direction. All helical piers must be galvanized, or corrosion protected. Again, the following can only be used if the lower terrace soils are suitably penetrated and develop the needed torque. Plates larger than 12 inches are not recommended due to anticipated penetration issues, unless proved otherwise by indicator piling.

Helical Pier Type	Inclination	Est. Length (ft)	Allowable Load* (kips)
10" and 12" Double with 3-1/2" pipe with threaded or sleeved and double bolted connection	Vertical	53-58+	40 (C), 36 (T)

* C – Compression T – Tension + - installation depth will vary and must fully penetrate the upper terrace soils

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Capacities for additional pier sizes and inclinations can be provided upon request. We recommend that we be retained to review pier support design and be called to the site to observe and document pier installation.

Drainage

The ground surface must be sloped to drain away from the building on all sides. A perimeter drain is required around all exterior foundations. The drain must consist of a two-foot width of drain rock encompassing a 4-inch diameter perforated pipe, all enclosed with a nonwoven filter fabric. The drain rock must have no more than 2% passing a #200 sieve and must extend to within one foot of the ground surface. The geosynthetic should be a Mirafi 160n or equivalent. One foot of low permeability soil (such as on-site silt) must be placed over the fabric at the top of the drain to isolate the drain from surface runoff. The drain must be tight-lined to a suitable discharge as determined by the civil engineer. Gutters must be maintained as free flowing.

Earthwork

Preparation - Prior to earthwork the site must be prepared by removal of any existing structures and utilities that conflict with new infrastructure. If utilities are encountered during site excavation, they must be rerouted away from the building area, or properly abandoned. Abandonment requires removal and backfill with granular structural fill, or full grouting with confirmation of grout at both ends of the conduit and a volume check for continuity.

Site preparation for earthwork may also require removal of existing fill to reach building or pavement subgrades. Fill north of the loop road is not expected to be suitable for fill as it was organic to depths of 7 to 9 feet in the 2013 test pits. Fill within the loop may be possible to reuse in dry summer conditions if properly moisture conditioned/dried to near optimum for compaction.

In the helipad pavement area and in other areas where cuts expose organic soils, it may be possible to stabilize the subgrade with the placement of fabric over geogrid capped with two feet of clean well graded crushed rock.

Removal of the fills must be done carefully to prevent disturbance of the underlying soils. We recommend using a smooth bucket excavator working on top of the material to be removed and loading into trucks supported on haul roads.

Stabilization and Soft Areas - After cuts are made and topsoil removed, the exposed soils must be evaluated. This can be done by the geotechnical engineer observing wheel rolling in dry conditions or probing in wet conditions. Soft areas will require over excavation and stabilization with a nonwoven separation geosynthetic and overlying grid, and backfill with well graded, angular crushed rock compacted as structural fill. The separation geosynthetic must consist of a Mirafi 801 or approved equivalent, and the grid a Hanes EGrid 2020 or equivalent.

Working Blankets and Haul Roads - Construction equipment must not directly traffic soils with more than trace silt as they are susceptible to disturbance when wet. Rock working blankets and haul roads placed over a stabilization geosynthetic in a thickened advancing pad can provide this protection.

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For working blanket and haul road rock, we recommend sound, angular, pit run or crushed basalt with no more than 6% passing a # 200 sieve. Working blankets must be at least 12 inches thick, and haul roads at least 18 inches thick, and can be placed in one lift over a Mirafi 801 separation fabric. Some repair of these elements must be expected.

Fill - Structural fill must consist of pit run rock less than 6 inches in nominal size compacted to 92% relative to ASTM D-1557 or to a dense state as observed by our geotechnical engineer, and must also pass a wheel roll. In wet conditions, this criteria can typically only be met by rock with less than 6% or less fines. The on-site silty angular gravel and sand fills may be for fill in dry conditions of late summer if properly moisture conditioned. Such fills must be placed in lifts no greater than 12 inches in loose thickness.

Cut Slopes - Cut slopes should not be made steeper than 3H:1V, and no closer than 25 feet from the planned buildings, and only after proposed cuts are submitted to us for stability evaluation.

It should be noted that the fill slope immediately east of the existing storage shed may deform and slump down in an earthquake, and may impact the shed. This may preclude the use of the shed for mechanical support equipment or other settlement sensitive contents.

Trenches – Utility trenches may encounter ground water seepage and severe caving at depth as encountered in the culvert installation excavations reported by Mike McEwan. Seepage was not encountered in our test pits but is expected to be perched at shallow depths in the wet season. Even above seepage levels, caving in the fill is expected and likely will be worse than the temporary short length cuts in the test pits. Proper shoring is required, with dewatering required if excavations encounter seepage. Increased backfill volumes are expected and must be included in the project budget and schedule. Trench base stabilization will likely be required for inverts where seepage is present. Stabilization with at least 12 inches of clean, well graded, angular pit run rock must be expected. Pipe bedding must be in accordance with the pipe manufacturers' recommendations. Trench backfill above the pipe zone must consist of well graded, angular crushed rock with no more than 7% passing a # 200 sieve. Trench backfill must be compacted to 92% relative to ASTM D-1557, with paving not occurring within one week of backfilling.

Utilities - The deformations in an earthquake may damage utilities, especially less ductile conduit or conduit with little tension capacity at the joints. The preceding differential movement in the ***Stability Analyses*** section can be used to evaluate utility performance, and consideration of flexible connections, alignment, materials, and allowance for deformation should be made. It would be prudent to include emergency power and communication systems contiguous with the reinforced mat or grade beam system of the building to reduce risk to emergency systems.

Pavement

Design - We have developed asphalt concrete pavement thickness at the site for 3 trucks per day (with a truck factor of 0.6) and a 20-year design life. These volumes can be revised if specific traffic data is available. Designs are also suitable to support a 75,000-pound fire truck. Our analyses are based on AASHTO methods and subgrade of undisturbed medium stiff silt or better native silt or fill having a resilient modulus of 3,000 psi. Construction will likely require protection and stabilization of subgrades

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as recommended in the ***Stabilization and Soft Areas and Working Blankets*** and ***Haul Roads*** sections of this report, and a Propex Geotex 801 (or equivalent) separation geosynthetic is required. Stabilization is expected to be needed particularly under the northern pavement areas where organic fill is expected. The results of our analyses based on these parameters are provided in the following table.

The main entry drive and any helicopter pad area should be underlain by a non-woven geosynthetic and two layers of geogrid, one located on top of the non-woven and one six inches up from it. This grid is intended to reduce the size of individual pavement cracks and vertical offsets to improve access after earthquake movement (the total cracking is expected to be the same).

Based on the results of our analyses we recommend a minimum of 3.0 inches of asphalt concrete (AC) over 12 inches of crushed rock base (CRB) in the main drive, helicopter landing, and any truck areas. Areas exposed to only car traffic can be constructed of 3 inches of AC over 8 inches of CRB. The rock sections will need to conform to haul roads and working blankets in the wet season.

Subgrade Preparation - The pavement subgrade should be prepared in accordance with the ***Earthwork*** recommendations presented in this report. All pavement subgrades will need to pass a proof roll prior to paving. Soft areas should be repaired by over excavating the areas, installing a separation geosynthetic and geogrid, and be brought to grade with well graded, angular crushed rock compacted as structural fill. For a separation geosynthetic we recommend a Propex Geotex 801 or equivalent, and the geogrid a Hanes Egrid 2020 or equivalent.

Base Rock and Asphalt Concrete - The recommended thicknesses are intended to be the minimum acceptable in dry conditions. Greater thicknesses are expected to be needed in wet conditions per the ***Earthwork, Stabilization*** sections in this report. Crushed rock should conform to ODOT base rock standards and have less than 6 percent passing the #200 sieve. Asphalt concrete should be compacted in lifts no greater than 3 inches in thickness to 91 percent of a Rice Density, or to 98 percent of the maximum density from a test strip.

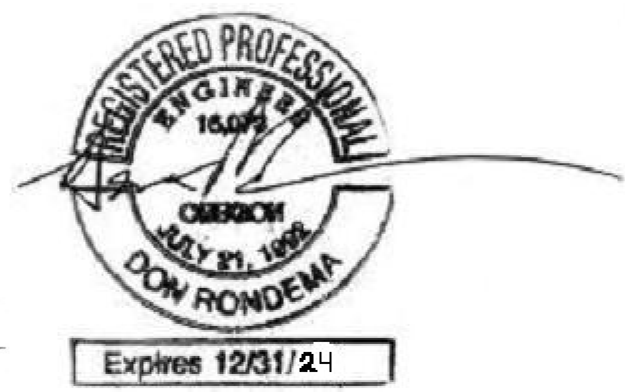
LIMITATIONS AND OBSERVATION DURING CONSTRUCTION

We have prepared this report for use by the City of Cannon Beach and members of their design and construction team for this project only. The information herein could be used for bidding or estimating purposes but must not be construed as a warranty of subsurface conditions. We have made observations only at the aforementioned locations and only at the stated depths. These observations do not reflect soil types, strata thicknesses, water levels or seepage that may exist between observations. We must be consulted to observe all foundation bearing surfaces, helical piers, proof rolling of slab and pavement subgrades, installation of structural fill, and any cut slopes. We must be consulted to review final design and specifications to see that our recommendations are suitably followed. If any changes are made to the anticipated locations, loads, configurations, or construction timing, our recommendations may not be applicable, and we must be consulted. The preceding recommendations must be considered preliminary, as actual soil conditions may vary. For our recommendations to be final, we must be retained to observe actual subsurface conditions encountered. Our observations will allow us to interpret actual conditions and adapt our recommendations if needed.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty, express or implied, is given.

We appreciate the opportunity to work with you on this project and look forward to our continued involvement. If you have any questions, please contact us.

Sincerely,



Don Rondema, MS, PE, GE
Principal

Attachments:

- Site Plan
- Guidelines for Classification of Soil and Rock
- Test Pit Logs
- Boring Logs
- Moisture Content
- Stability Sections
- A-A Stability Model
- A-A Static Stability
- A-A yield acceleration
- A-A example displacement
- B-B static stability
- B-B yield acceleration
- Horning Geologic Map Excerpt
- City Geological Map Excerpt
- DOGAMI Bulletin 74 Excerpt
- SLIDO landslide susceptibility
- DOGAMI Tsunami Map Excerpt
- ASCE 7-16 hazard tool output
- Seismic Hazard Investigation

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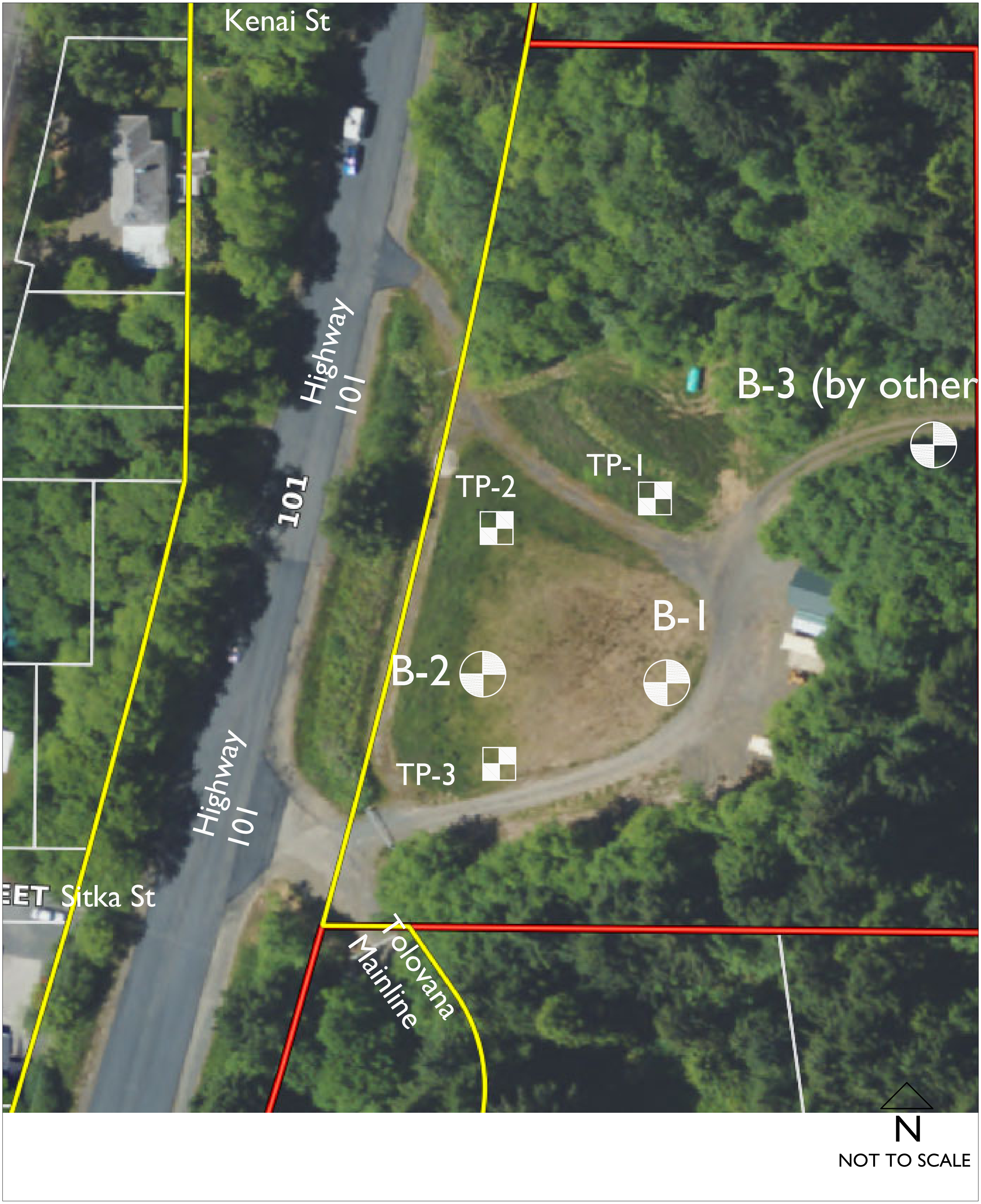

BASE PHOTO FROM 2022 AERIAL

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SITE PLAN

cannon-22-4-gi

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil	
Relative Density	**Standard Penetration Resistance (N-values) blows per foot**
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils		
Consistency	**Standard Penetration Resistance (N-values) blows per foot**	**Torvane Undrained Shear Strength, tsf**
very soft	0 - 2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Grain-Size Classification	
Description	**Size**
Boulders	12 - 36 in.
Cobbles	3 - 12 in.
Gravel	¼ - ¾ in. (fine)
	¾ - 3 in. (coarse)
Sand	No. 200 - No. 40 Sieve (fine)
	No. 40 - No. 10 sieve (medium)
	No. 10 - No. 4 sieve (coarse)
Silt/Clay	Pass No. 200 sieve

Modifier for Subclassification	
Adjective	**Percentage of Other Material In Total Sample**
Clean/Occasional	0 - 2
Trace	2 - 10
Some	10 - 30
Sandy, Silty, Clayey, etc.	30 - 50

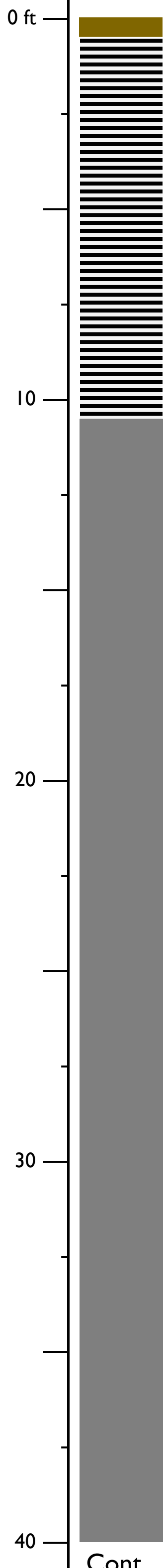
GUIDELINES FOR CLASSIFICATION OF ROCK

Scale of Rock Hardness		
Hardness	**Description**	**Definition**
RH-0	Very Soft	For plastic material only
RH-1	Soft	Carved or gouged with a knife
RH-2	Moderately Hard	Scratched with a knife
RH-3	Hard	Difficult to scratch with a knife
RH-4	Very Hard	Rock scratches metal; rock cannot be scratched with a knife

Terms used to Describe the Degree of Weathering	
Description	**Definition**
Severely Weathered	Rock decomposed; thorough discoloration; all fractures extensively coated with clay, oxides, or carbonates
Moderately Weathered	Intense localized discoloration of rock; fracture surfaces coated with weathering minerals
Little Weathered	Slight and intermittent discoloration of rock; few stains on fracture surfaces
Fresh	Rock unaffected by weathering

Relation of RQD and Rock Quality	
Rock Quality Designation (RQD), %	**Description of Rock Quality**
0 - 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

Descriptive Terminology for Joint Spacing	
Spacing of Joints	**Description**
< 2 in	Very Close
2 in - 1 ft	Close
1 ft - 3 ft	Moderately Close
3 ft - 10 ft	Wide
> 10 ft	Very Wide



Soil and Rock Description

Medium stiff, dark gray SILT FILL, with trace gravel; moist.

Becomes with trace to some clay

Very soft to stiff, gray SILT, with trace to some clay; moist. (marine terrace inferred as ancient slide debris from inconsistent structure).

very stiff; wet.

very soft, sandy SILT; wet.

soft to medium stiff with some wood debris.

Samples and Data

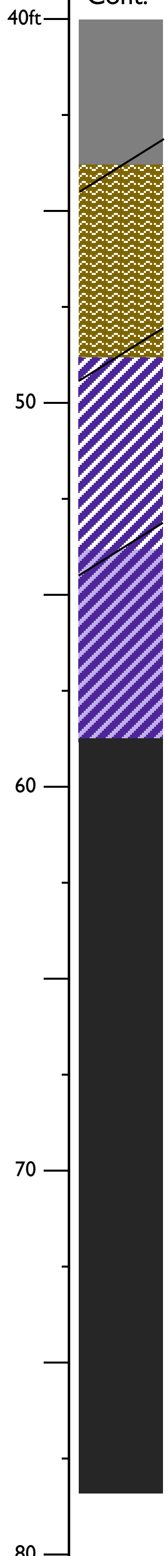
N_{85}	w
6	w = 34%
5	w = 37%
5	w =31%
9	w = 37%
22	w = 32%
0	w = 45%
5	w = 157%
3	w = 148%

N_{85} = SPT blowcount
w = moisture content
f = percent fines
γ_d = dry unit weight

Cont.

Soil and Rock Description

Samples and Data



Depth	Soil and Rock Description	Sample	Data
		5	w = 156%
	stiff SILT with some fine sand and fine organics. Sedimentary structure -not slide debris. (inferred as older marine terrace).	16	w = 130%
50	Medium stiff, gray/blue gravelly SILT (severely weathered siltstone with gravel sized clasts)	6	w = 45%
	Soft to moderately hard (RH1-2) little weathered gray SILTSTONE	53	w = 15%
60	Hard, RH-3, little weathered, closely fractured dark gray/black BASALT. Coring attempt unsuccessful due to fracturing. Tricone bit advanced roughly 8 feet an hour.	50/ 6”	

NOTES: Boring completed to 78 feet on Sep 7, 2023. Backfilled with bentonite chips.

N_{85} = SPT blowcount
w = moisture content
f = percent fines
γ_d = dry unit weight

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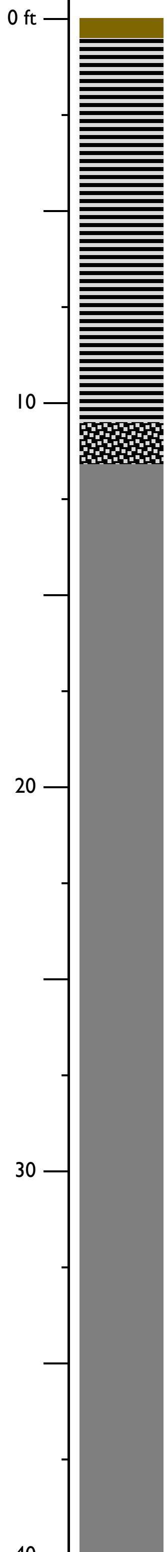
BORING B-1 (cont.)

Cannon 22-4-gi

Soil and Rock Description

Samples and Data



Soil and Rock Description	Samples and Data	
Medium dense, fine gray SAND FILL, with trace silt and some gravels and	14	w = 17%
becomes silty	24	w = 13%
	12	w =17%
Medium dense, angular black GRAVEL FILL; moist.		
Medium stiff to stiff, gray SILT, with trace sand and trace to some clay; moist (marine terrace inferred as ancient slide debris from inconsistent structure).	12	w = 29%
	19	NR
Becomes medium stiff to stiff with occasional wood debris.; wet	11	w = 29%
	8	w = 39%
	7	w = 75%

Cont.

N_{85} = SPT blowcount
w = moisture content
f = percent fines
γ_d = dry unit weight

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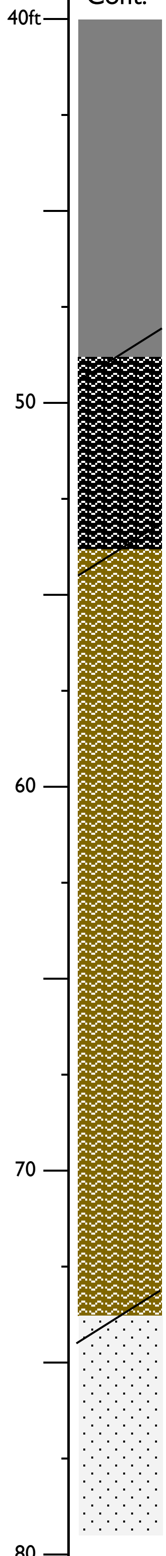
BORING B-2

Cannon 22-4-gi

Cont.



Soil and Rock Description

very soft to soft with some wood debris.

medium stiff with some wood debris.

Very soft gray SILT with trace fine sand and organics; moist. (undisturbed sedimentary structure, non-landslide, inferred as top of older marine terrace).

Stiff, brown, sandy SILT, with some organics; moist. (Older marine terrace - non landslide).

Very dense fine gray SAND; wet. (Older marine terrace).

Cont.

Samples and Data

0	w = 54%
5	w =197%
0	w = 48%
11	w = 177%
8	w = 100%
15	w = 81%
9	w = 86%
86	w = 17%

N_{85} = SPT blowcount
w = moisture content
f = percent fines
γ_d = dry unit weight

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BORING B-2 (cont.)

Cannon 22-4-gi

Cont.

80

90

80

Soil and Rock Description

Hard, RH-3, little weathered, moderately fractured dark gray/black BASALT

Tricone bit advanced roughly 6 feet per hour.

NOTES: Boring completed to 100 feet on Sep 8, 2023. Backfilled with bentonite chips.

Samples and Data

50/ 5" w = 18%

50/ 0"

N_{85} = SPT blowcount
w = moisture content
f = percent fines
γ_d = dry unit weight

Test Pit #	Depth (ft)	Soil Description
		Explorations completed on September 6, 2023 with a track mounted excavator.
TP-1		**Location:** NE portion of site.
		Surface conditions: Short grass, weeds.
	0 – 5	Loose, light brown gravelly SAND FILL, with trace silt and asphalt debris; dry
	5 - 9	Medium dense, light brown gravelly SAND FILL, with occasional boulders; dry.
	9 – 11	Medium stiff, brown sandy SILT FILL, with some gravels and cobbles and trace organics; moist.
	11 – 15	Stiff, brown SILT, with some sand and siltstone gravels and cobbles; moist.
		Minor caving beneath 5'. No seepage.
TP-2		**Location:** NW portion of site.
		Surface conditions: Short grass, weeds.
	0 – 5	Loose, light brown gravelly SAND FILL, with trace silt and asphalt debris; dry
	5 – 11	Medium stiff, brown sandy SILT FILL, with some gravels and cobbles and trace clay; moist.
	11 – 15	Stiff, brown SILT, with some sand and siltstone gravels and cobbles; moist.
		Minor caving beneath 5'. No seepage.
TP-3		**Location:** SW portion of site.
		Surface conditions: Short grass, weeds.
	0 – 5	Loose, light brown gravelly SAND FILL, with trace silt and asphalt debris; dry
	5 – 11	Medium stiff, brown sandy SILT FILL, with some gravels and cobbles and trace organics; moist.
	11 – 15	Stiff, brown SILT, with some sand and siltstone gravels and cobbles; moist.
		Minor caving beneath 5'. No seepage.

Exploration	Depth, ft	Moisture Content
TP-1	5.0	34%
TP-1	8.0	41%
TP-2	4.0	21%
TP-2	7.0	9%
TP-2	9.0	28%
TP-2	14.0	24%
TP-3	8.0	94%
TP-3	13.0	15%
B-1	2.5	34%
B-1	5.0	37%
B-1	10.0	31%
B-1	15.0	37%
B-1	20.0	32%
B-1	25.0	45%
B-1	30.0	157%
B-1	35.0	148%
B-1	40.0	156%
B-1	45.0	130%
B-1	50.0	45%
B-1	55.0	15%
B-2	2.5	17%
B-2	5.0	13%
B-2	10.0	17%
B-2	15.0	29%
B-2	25.0	29%
B-2	30.0	39%
B-2	35.0	75%
B-2	40.0	54%
B-2	45.0	197%
B-2	50.0	48%
B-2	55.0	177%
B-2	60.0	100%
B-2	65.0	81%
B-2	70.0	86%
B-2	75.0	17%
B-2	80.0	18%

N

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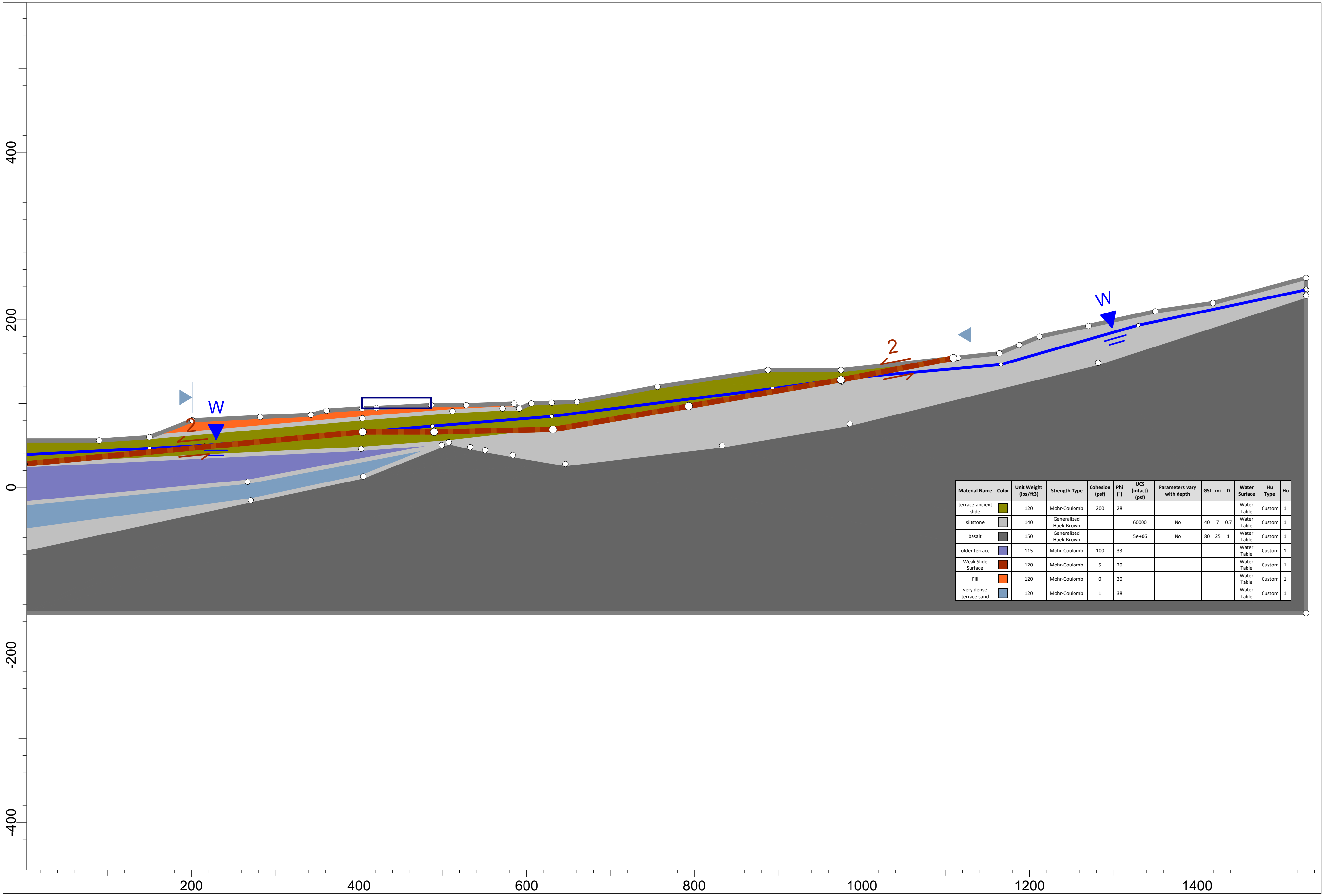
STABILITY SECTIONS

cannon-22-4-gi



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	UCS (intact) (psf)	Parameters vary with depth	GSI	mi	D	Water Surface	Hu Type	Hu
terrace-ancient slide		120	Mohr-Coulomb	200	28						Water Table	Custom	1
siltstone		140	Generalized Hoek-Brown			60000	No	40	7	0.7	Water Table	Custom	1
basalt		150	Generalized Hoek-Brown			5e+06	No	80	25	1	Water Table	Custom	1
older terrace		115	Mohr-Coulomb	100	33						Water Table	Custom	1
Weak Slide Surface		120	Mohr-Coulomb	5	20						Water Table	Custom	1
Fill		120	Mohr-Coulomb	0	30						Water Table	Custom	1
very dense terrace sand		120	Mohr-Coulomb	1	38						Water Table	Custom	1

Section A-A.slmd

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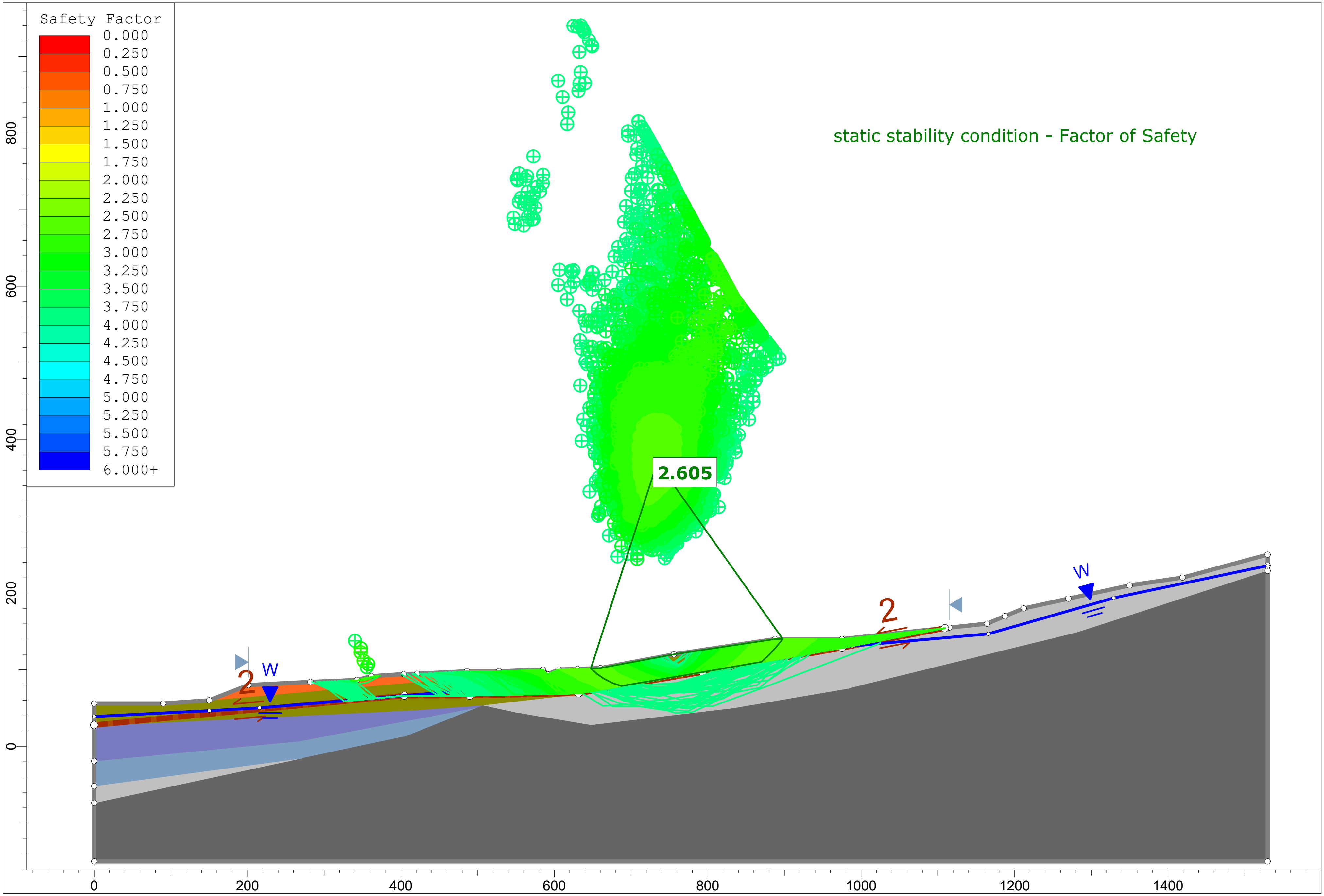

Section A-A.slmd

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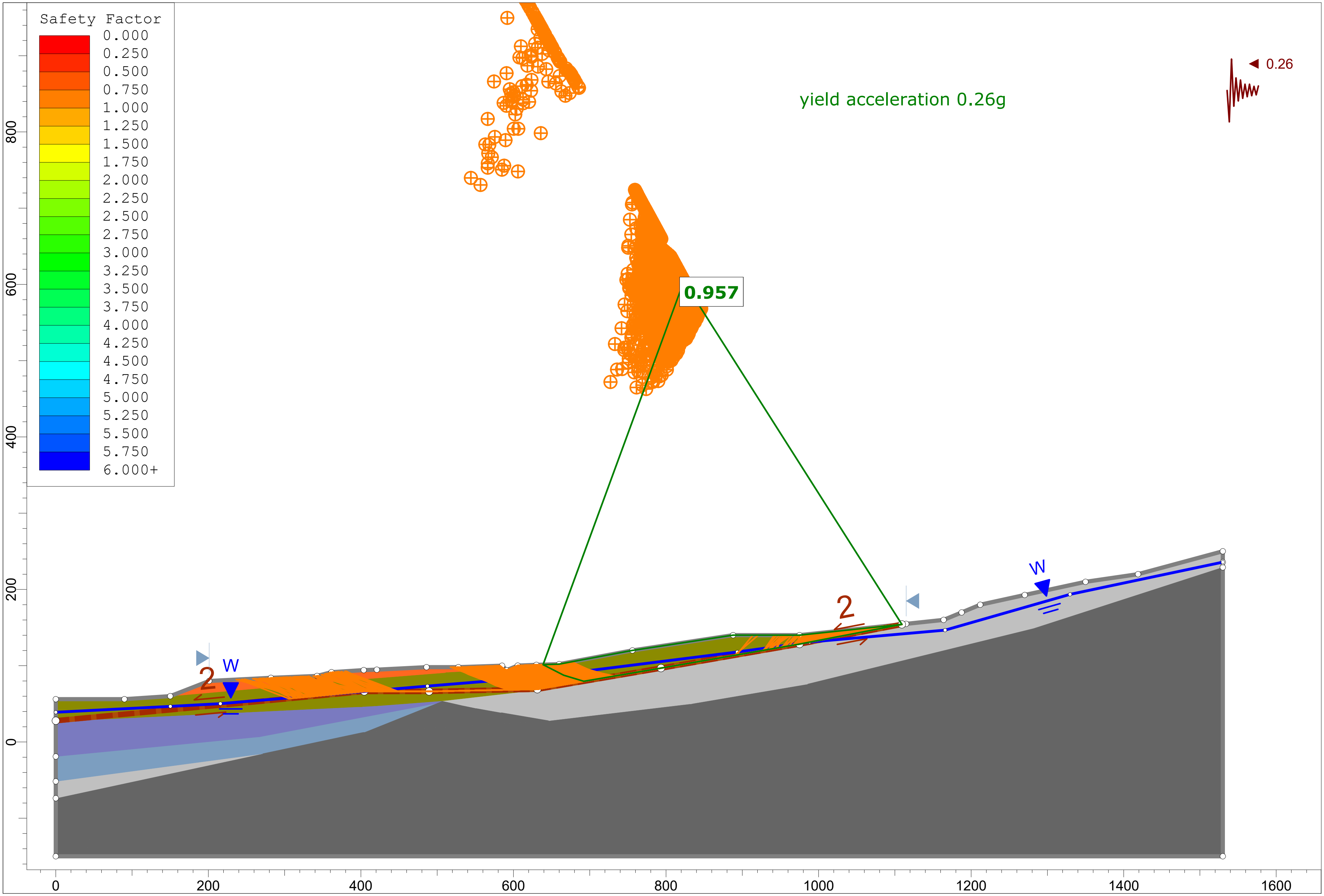

Section A-A 0p26g weak surface at spt0.slmd

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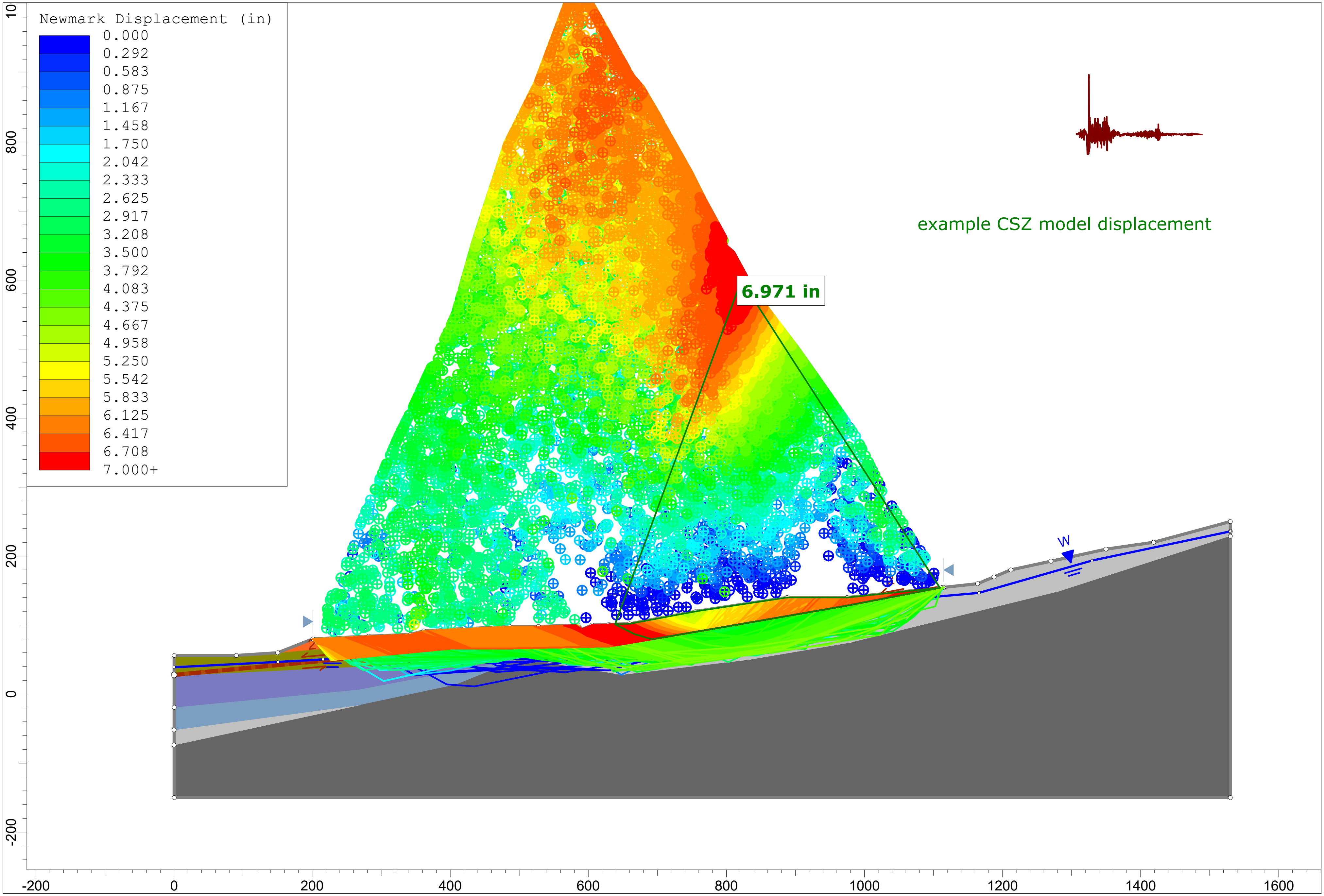

Section A-A Ky newmark displacement - CSZ model

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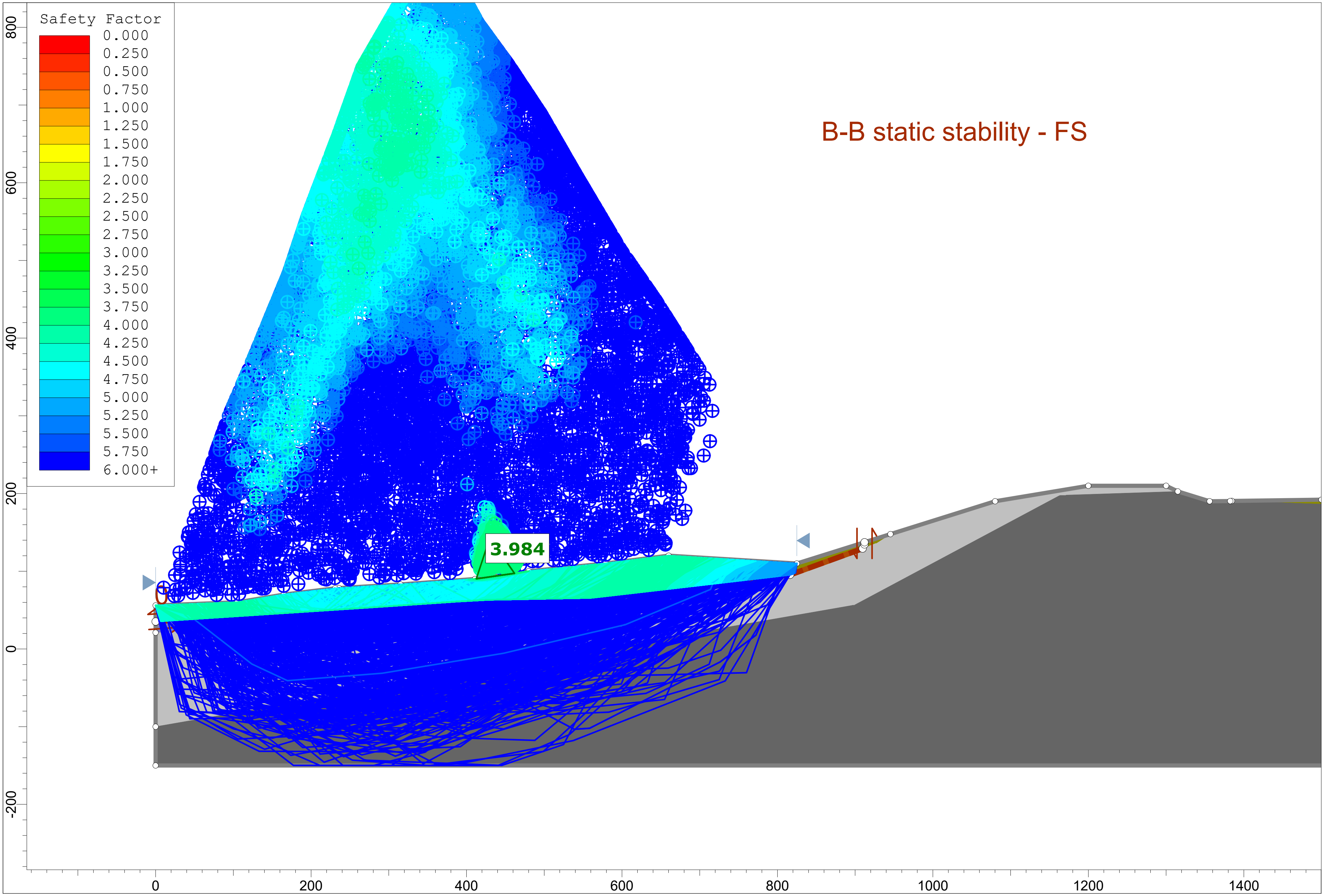

Section B-B.slmd

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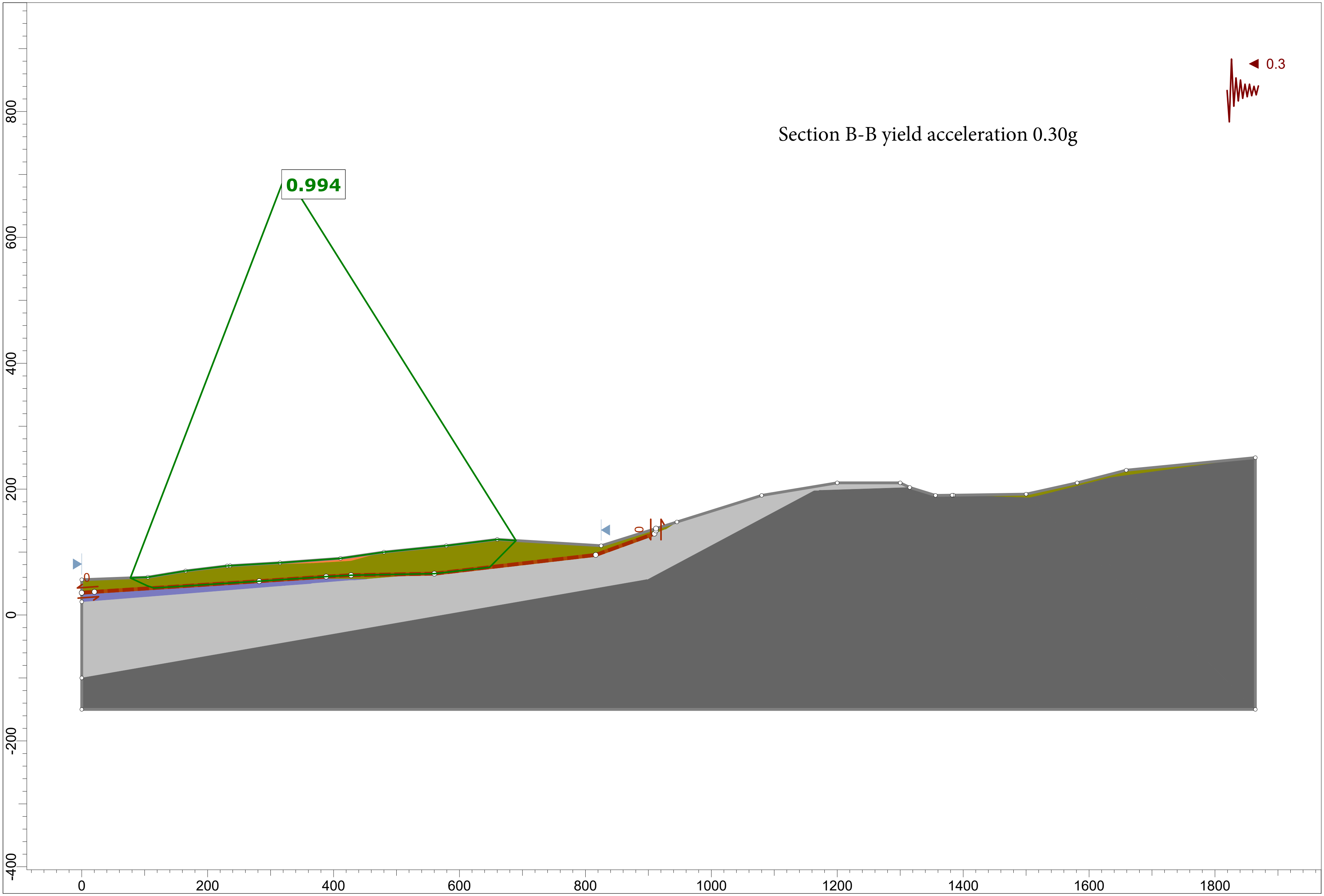

Section B-B.slmd

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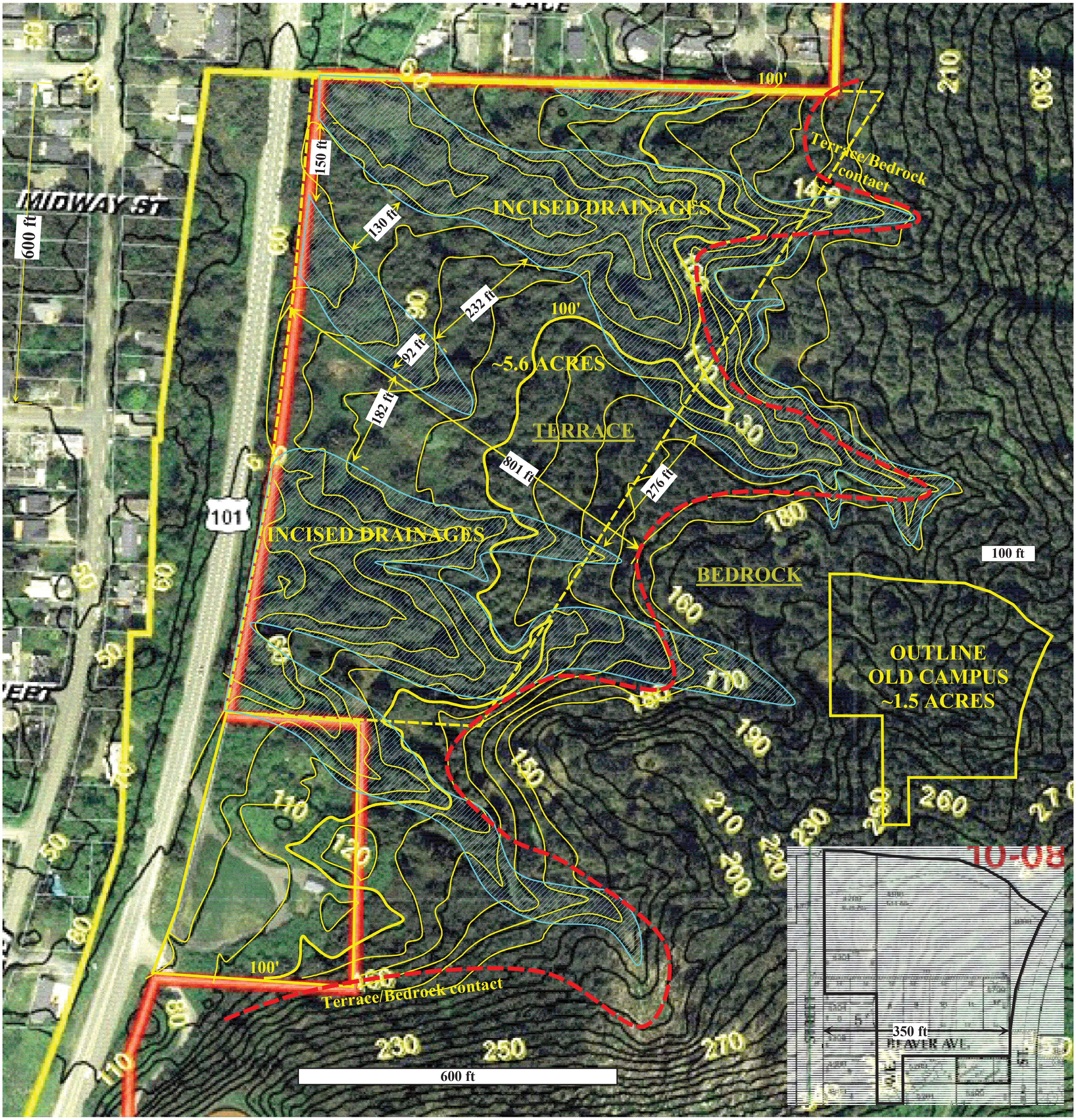
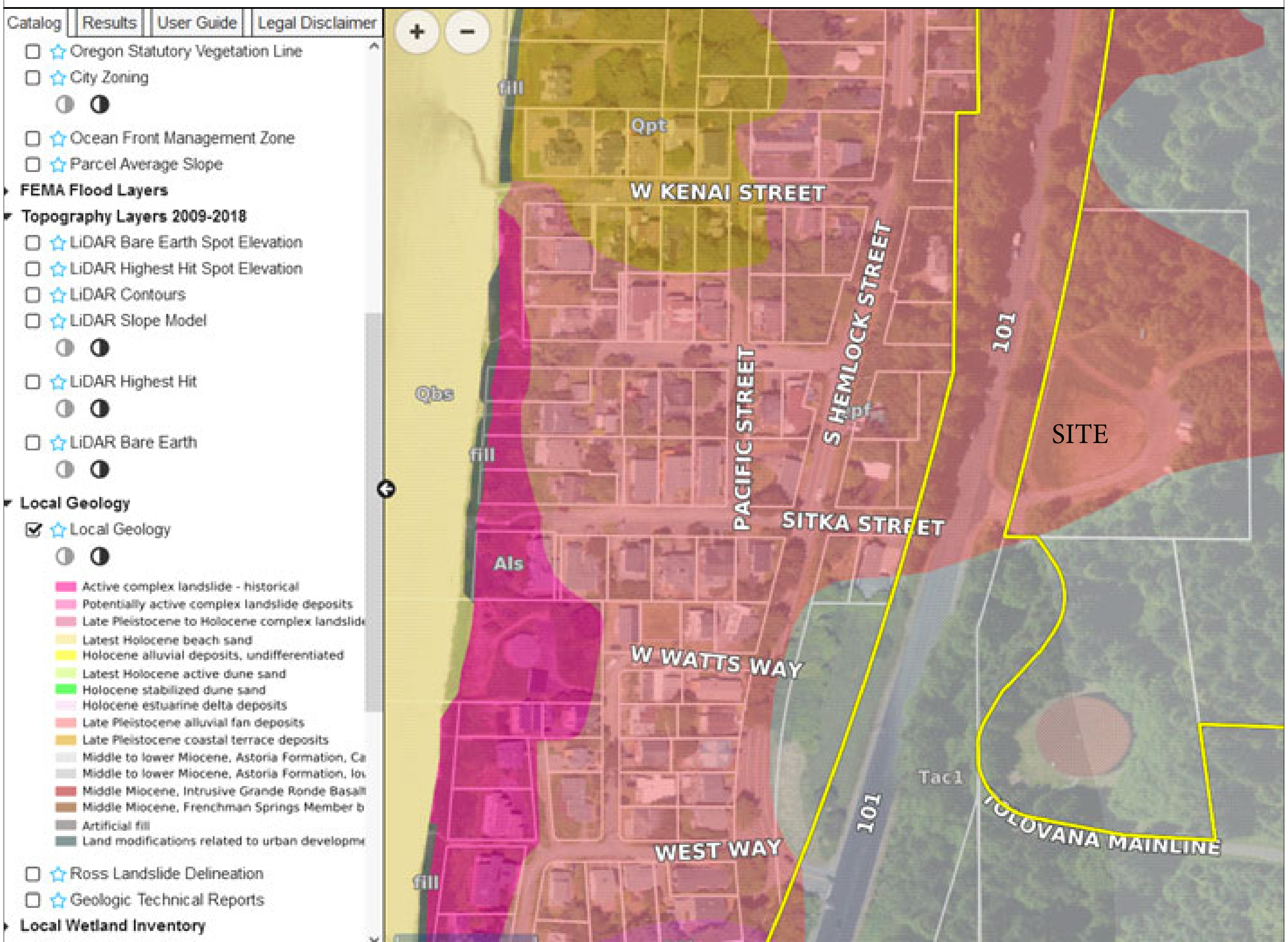

MIDWAY ST
600 ft
150 ft
130 ft
232 ft
92 ft
182 ft
801 ft
276 ft
100'
INCISED DRAINAGES
~5.6 ACRES
TERRACE
Terrace/Bedrock contact
INCISED DRAINAGES
BEDROCK
100 ft
OUTLINE
OLD CAMPUS
~1.5 ACRES
101
Terrace/Bedrock contact
600 ft
350 ft
BEAVER AVE
10-08
210
230
180
160
170
150
110
120
260
270
250
200
220
190
140
130
90
60
50
80



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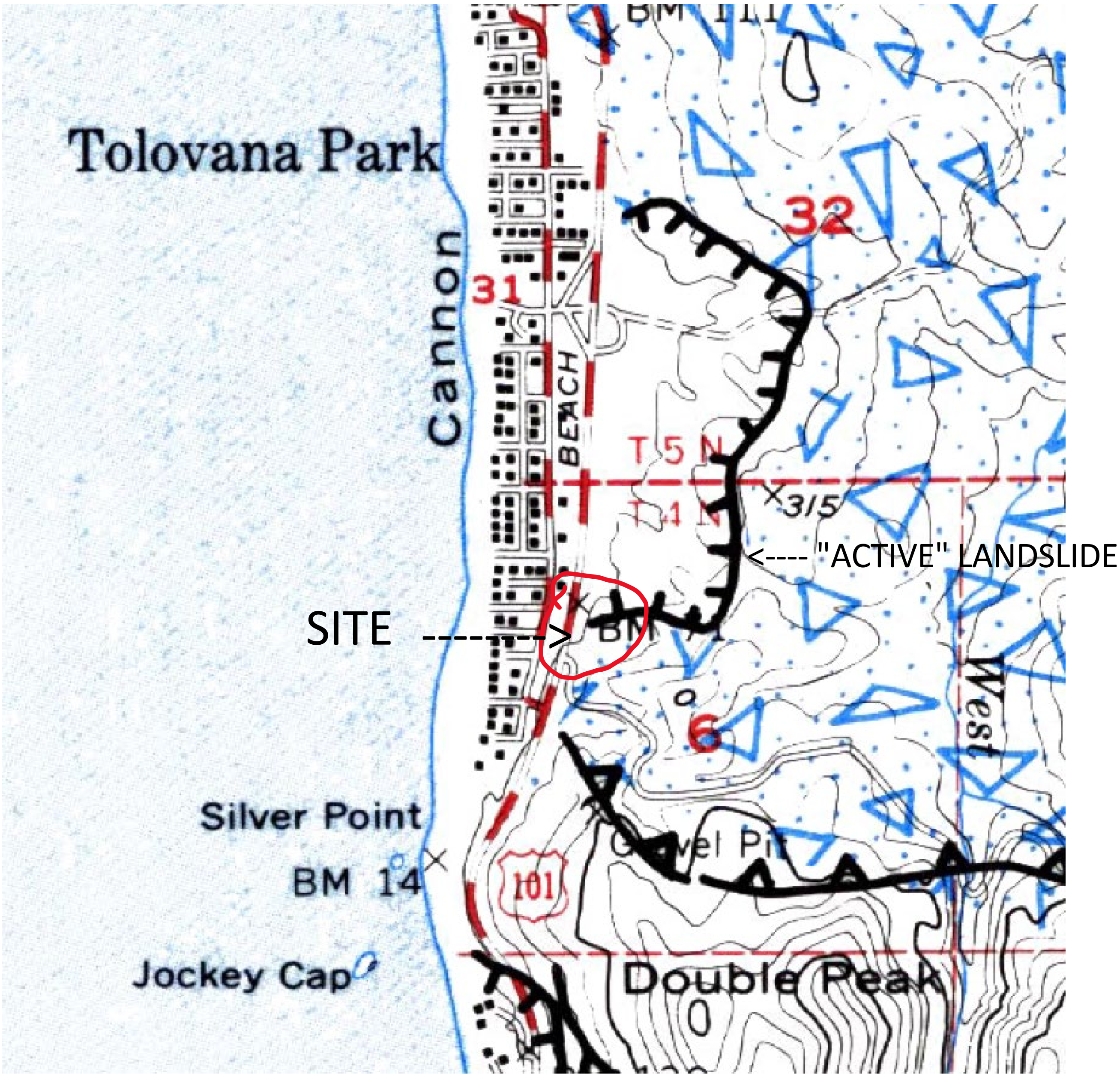
CITY GIS GEOOGICAL MAPPING

cannon-22-4-gi


Tolovana Park
Cannon
BEACH
31
32
T 5 N
T 4 N
315
<---- "ACTIVE" LANDSLIDE
SITE ---------->
BM
6
West
Silver Point
BM 14
101
Gravel Pit
Jockey Cap
Double Peak

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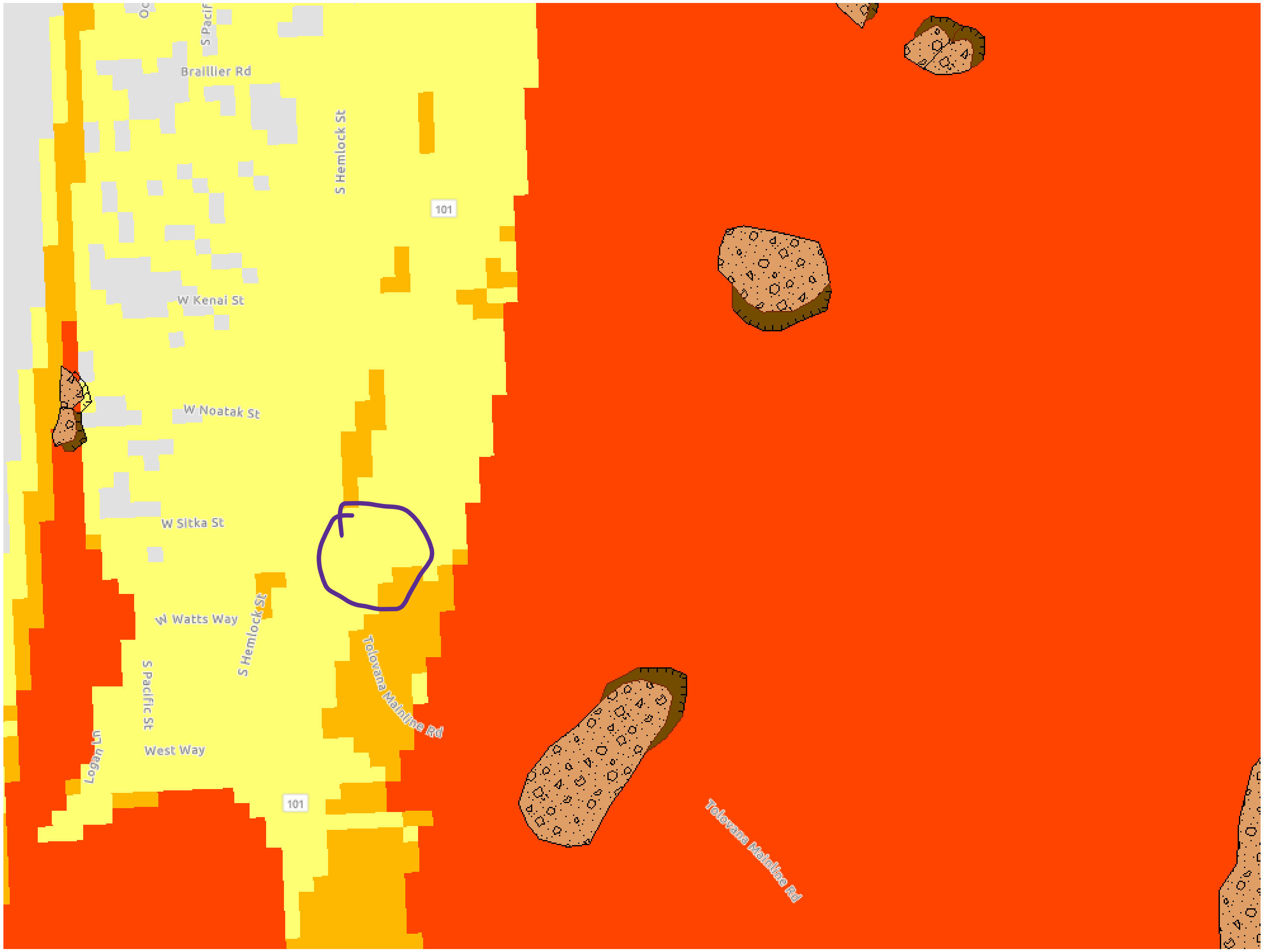
Oregon SLIDO Landslide Susceptibility



BASE PHOTO FROM 2022 AERIAL

Geotech Solutions Inc.

DOGAMI TSUNAMI MAPPING

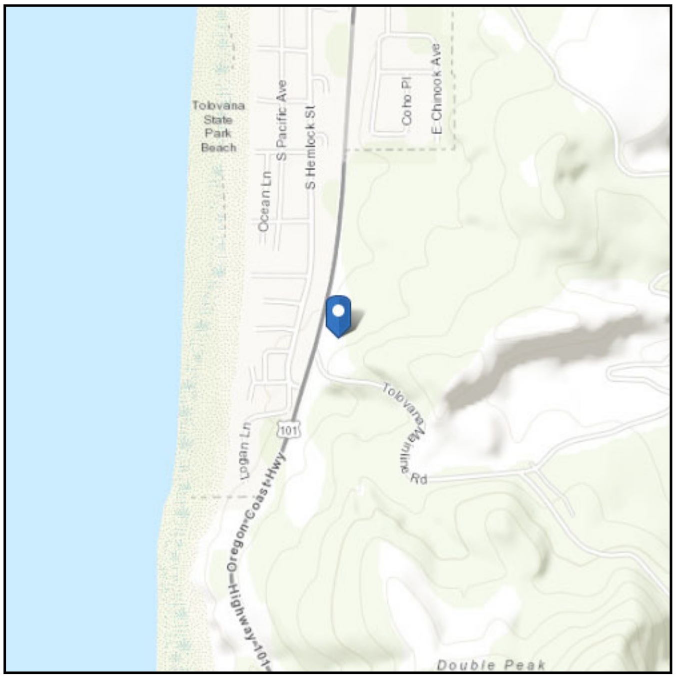
cannon-22-4-gi

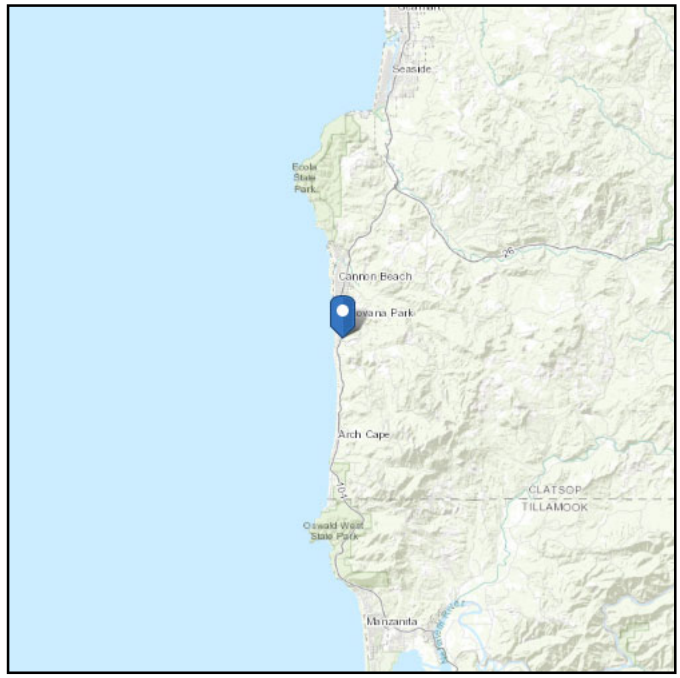
ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: IV
Soil Class: D - Stiff Soil

Latitude: 45.86268
Longitude: -123.958819
Elevation: 103.74040760446984 ft (NAVD 88)





AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

Site Soil Class: D - Stiff Soil

Results:

S_S :	1.312	S_{D1} :	N/A
S_1 :	0.688	T_L :	16
F_a :	1	PGA :	0.661
F_v :	N/A	PGA_M :	0.727
S_{MS} :	1.312	F_{PGA} :	1.1
S_{M1} :	N/A	I_e :	1.5
S_{DS} :	0.875	C_v :	1.362

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Thu Sep 21 2023

Date Source: USGS Seismic Design Maps

The ASCE 7 Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE 7 standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this Tool or the ASCE 7 standard.

In using this Tool, you expressly assume all risks associated with your use. Under no circumstances shall ASCE or its officers, directors, employees, members, affiliates, or agents be liable to you or any other person for any direct, indirect, special, incidental, or consequential damages arising from or related to your use of, or reliance on, the Tool or any information obtained therein. To the fullest extent permitted by law, you agree to release and hold harmless ASCE from any and all liability of any nature arising out of or resulting from any use of data provided by the ASCE 7 Hazard Tool.

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SEISMIC HAZARD INVESTIGATION

General

We have evaluated earthquake hazards in accordance with the degree of complexity of the proposed project and the site per SOSSC guidelines. This included literature and map review, as well as site specific subsurface investigations and analyses described in detail in the preceding report. Based on this evaluation, tsunami inundation hazards are low. Overall ground motion and amplification hazards are moderate and can be accommodated with code based design and the recommendations in our report. Liquefaction hazards are low due to the stiff and cohesive nature of the native site soils. Risk of on-site fault rupture is low. The risk of dynamic slope instability for the east-west terrace deposit sections across the site is high, with moderate deformations, as discussed in detail in the report text. The risk of instability for the southeast slope is moderate. A summary of the basis for these opinions is included herein.

Seismic Sources and Design Earthquake

Three earthquake types can induce ground motions at the site. These include local crustal earthquakes, and both CSZ intraplate and interface earthquakes. Local crustal earthquakes may occur from northwest trending faults in the region, most possibly from the Gales Creek or Tillamook Bay fault zones over 20 miles from the site, or possibly from small faults that are as close as 3.5 miles that are present in the accretionary wedge. These are shown on the attached fault map (USGS Quaternary fault database). However, these local crustal faults are considered a low hazard. CSZ intraplate earthquakes are presumed possible within the subducted Juan de Fuca plate, with estimated magnitudes of 7.0 to 7.5. These earthquakes are analogous to the 2001 Nisqually earthquake near Olympia as well as other large earthquakes historically beneath southern Puget Sound and inferred beneath the southern Oregon coast. The expected depth of these presumed earthquakes of 40 to 60 km, and when coupled with low seismicity in western Oregon they present a moderate hazard. A CSZ interface earthquake presents a high hazard for the site area and is the controlling design earthquake, as evidenced by USGS hazard de-aggregations (USGS OFR 2008-1128). Such an event has an expected magnitude of 8.7 to 9.1 and recurrence intervals roughly from 100 to 1100 years. A magnitude $M_w = 8.7$ is expected to correspond to an average 10% chance of being exceeded in 50 years, with $M_w = 9.0$ corresponding to 2% in 50 years. It is possible that such earthquakes could occur with hypo-central distances of 20 to 40 kilometers. Duration of strong ground motion is expected to be several minutes, and repeated cycles of horizontal ground acceleration are expected in the 0.35 to 0.50g range, with PHGA listed as 0.73 g by the USGS.

Tsunami Inundation

Based on a review of tsunami inundation elevations on maps (DOGAMI TIM-Clat-09, 2013 – excerpt attached) the proposed facilities will be located above tsunami inundation elevations of roughly 80 feet which is west of Highway 101 in this location. The risk of tsunami inundation is low.

Amplification

Amplification hazards at the site are moderate based on the fundamental period of the soil column as derived from its stiffness and depth. Based on the site-specific conditions encountered, the mapped

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units, and the low-rise building planned, the amplification hazard at the site is accommodated by code level design consistent with our preceding report recommendations.

Liquefaction

The liquefaction hazard for the site is low primarily due to the cohesive nature of the native site soils. Although non-cohesive sand was present in the west boring B-2 at depths of 73-82 feet, this sand is very dense and not susceptible to liquefaction.

Fault Rupture

No faults are mapped as crossing the site (USGS Quaternary fault database), with the nearest Quaternary fault mapped as within 4 miles to the west-northwest within/below accretionary wedge sediment. The Gales Creek fault zone is the next nearest fault located roughly 16 miles to the east-southeast. Interface earthquakes from the CSZ are offshore and buried nearer shore, and intraplate CSZ earthquakes are deep within the subducted plate. Therefore, the hazard from potential fault rupture on-site is low.

Earthquake Induced Slope Instability

The site is mapped at the margin of a mapped landslide noted as "active" in DOGAMI Bulletin 74 which includes a broad scale perspective (excerpt attached). Site inclinometer readings from 2020-2023 indicate no site movement during that time, and no site indications of active instability have been noted in our site reconnaissance in 2013 and in 2023 for this report. Previous reports for the Southwind site also indicate that the area is not undergoing active movement.

The site subsurface below depths of roughly 43-48 feet includes marine terrace with variable structure, intact organic debris, and characteristics of landslide deposition. This deposit is susceptible to landslide movements at yield accelerations calculated to be above 0.26g, well below the design earthquake threshold. This would categorize the general earthquake instability risk as high, although deformations were analyzed to be moderate. Based on our interaction with the structural engineer, this risk can be accommodated for functional design by proper structural engineering that addresses the quantified deformations and foundation approaches and parameters detailed in the text of this report's ***Foundations*** section.

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