Last Chance Grade Permanent Restoration Project

Preliminary Geotechnical Report

Submittal SUB-051

December 2023 – FINAL

EA# 01-0F280

Project EFIS# 0115000099

Del Norte County, U.S. 101,

PM 12.7/16.5

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ACRONYMS AND ABBREVIATIONS

APEFZ	Alquist-Priolo Earthquake Fault Zone
ARS	Acceleration Response Spectrum
ASBS	Areas of Special Biological Significance
ATV	acoustic televiewer
BGS	below the ground surface
CalGEM	California Geologic Energy Management Division
Caltrans	California Department of Transportation
CBC	California Building Standards Code
CBSC	California Building Standards Commission
CGS	California Geological Survey
CIDH	cast-in-drilled-hole
CIP	cast-in-place
CSZ	Cascadia Subduction Zone
CY	cubic yards
DEM	digital elevation model
DWR	California Department of Water Resources
EDAS	Engineered Deformation Absorption System
EF	Large Earthflow Complex
ERS	earth retaining system
ESL	Environmental Study Limit
GSI	Geological Strength Index
H:V	horizontal to vertical
HEL	USDA Highly Erodible Land
HPGA	horizontal peak ground acceleration
HSA	Hydrologic Sub-Area
IGM	Intermediate Geo-Materials
LCG	Last Chance Grade
LiDAR	Light Detection and Ranging
M	Mean Earthquake Moment Magnitude
MSE	mechanically stabilized earth

MTD	Caltrans Memo to Designers
NCHRP	National Cooperative Highway Research Program
NLCG	North Last Chance Grade Complex
OG	original ground
OMC	Operations and Maintenance Center
OTV	optical televiewer
PA&ED	Project Approval and Environmental Document
PG	profile grade
PGA	peak ground acceleration
PGR	Preliminary Geotechnical Report
PM	post mile
Project	Last Chance Grade Permanent Restoration Project
RMR	Rock Mass Rating
RQD	Rock Quality Designation
RW	retaining wall
RWQCB	Regional Water Quality Control Board
SEE	Safety Evaluation Earthquake
SEM	Sequential Excavation Method
SFRDHA	Surface Fault Rupture Displacement Hazard Analysis
SI	slope inclinometer
SLCG	South Last Chance Grade Complex
SPT	standard penetration test
SWRCB	State Water Resources Control Board
TBM	tunnel boring machine
TDR	time domain reflectometry
UCERF3	Uniform California Earthquake Rupture Forecast, Version 3
U.S. 101	U.S. Highway 101
USBR	U.S. Bureau of Reclamation
USDA	United States Department of Agriculture
USGS	United States Geological Survey
USACE	United States Army Corps of Engineers

VWP	vibrating wire piezometer
WC	Wilson Creek Complex

1 INTRODUCTION

The Last Chance Grade (LCG) Permanent Restoration Project (Project) is located on a section of U.S. Highway 101 (U.S. 101) known as Last Chance Grade in southern Del Norte County, California. It is approximately 10 miles south of Crescent City, between post miles (PMs) 12.7 and 16.5. The location of the project is shown on Plate 1.

The purpose of the project is to develop a long-term solution to the instability and potential roadway failure at LCG, which has been progressively sliding towards the Pacific Ocean since the roadway was first constructed. Due to continual road deformation resulting from slope movement, ongoing construction and maintenance activities are necessary to keep U.S. 101 open to the traveling public. The Project is considering Alternatives X and F to provide a more reliable connection, reduce maintenance costs, and protect the economy, natural resources, and cultural landscapes.

1.1 Report Purpose and Scope

This Preliminary Geotechnical Report (PGR) has been prepared on behalf of the California Department of Transportation (Caltrans) in support of the Project Approval and Environmental Document (PA&ED) for the Project (Caltrans Project ID 0115000099-EA 01-0F280). The PGR presents preliminary geotechnical analyses and recommendations for cut slopes, embankments, earthwork, landslide and rockfall mitigation, tunneling, underground drainage, sub-excavation, and other geotechnical conditions that may affect the design and construction.

1.2 Assumptions and Definitions

The study areas for this PGR are associated with the preliminary alignment layouts (plan and profile) for only Alternatives X and F following the decision by Caltrans to continue with just those two alternatives. The preliminary alignment layouts and infrastructure features currently planned along each of the two alternatives were developed by the engineering design team during the summer of 2021 and advanced during 2022 and 2023. Alignments and features considered in this report are current as of October 26, 2023. The associated submittals and documents presenting information developed by the engineering designers are included in the References section of this PGR.

It has been assumed that repairs will be required to maintain the proposed structures.

Other assumptions that have been used in this study related to interpretation of Caltrans performance expectations and interpretation of preliminary design information are described in the text sections below. The various assumptions will require verification and/or potential updating during follow-on design phases, and for preparation of the preliminary and final Plans, Specifications, and Estimates (PS&E) for the project. This report is intended for use by Caltrans and the project design engineers and environmental staff during the current PA&ED phase.

Adjustments to alignment locations and details considered in this preliminary report could be implemented, if necessary, to mitigate potential environmental impacts.

Material is called "rock" or "bedrock" if it has a geologic formation name and the original geologic structure can be discerned, following convention for California (Wills, 2000).

1.3 Limitations

This report presents preliminary geotechnical information for the purpose of advancing the two selected alternatives, Alternative X and Alternative F, through completion of the (PA&ED). It does not address the “No-Build Alternative.”

Rock, soil, and groundwater conditions were observed and interpreted at the exploration locations only, which are limited in number, and conditions may change with time. Conditions are expected to vary between the exploration locations, and seasonal fluctuations in the groundwater levels are expected to occur due to variations in rainfall. Conclusions, opinions, interpretations, and/or recommendations are based on a limited number of observations and data. It is possible that conditions could vary significantly between or beyond the data evaluated.

This preliminary report is subject to revisions pending receipt of updated alignment and geotechnical data. The conceptual designs contained in this report were developed to be consistent with the geotechnical conditions presumed along the alignments, which are in turn based on a limited number of borings and interpolations between these borings. Since additional subsurface exploration and geotechnical monitoring (instrumentation) data will be collected and analyzed during subsequent project design phases, the associated interpretations, conclusions and/or recommendations presented in this preliminary report are subject to change at a later date. The same will apply if there are significant design changes to proposed infrastructure features, or plans and profiles for alignment Alternatives X and/or F.

The HNTB geotechnical team makes no other representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, plans, specifications, opinion, or instrument of service provided. This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of the geotechnical profession practicing in the same locality, under similar conditions and at the date the services are provided.

1.4 Project Description – Alternative X

1.4.1 General Description

Alternative X would involve reengineering a 1.6-mile-long section of the existing highway to minimize the risk of landslides. Main project components would include 1.6 miles of retaining walls along the roadway, an underground drainage system to help reduce landslide risk, and strategic eastward retreats from the existing roadway.

1.4.2 Alignment and Component Locations

The area of improvement would begin at PM 14.3 and would conform to the existing highway at PM 15.9.

Figure 1 is a schematic representation of the general location and features of Alternative X. The layout of Alternative X and the underground drainage system is shown in Plates 2a and 2b.

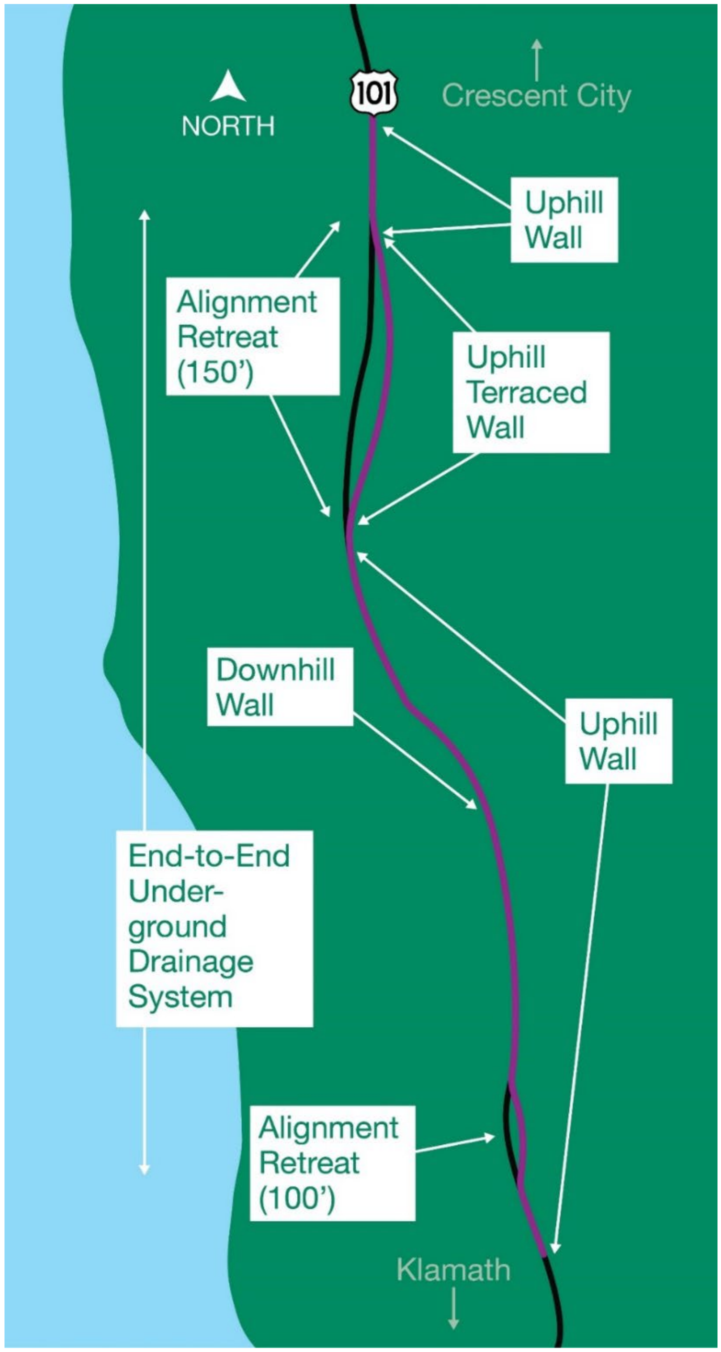


Figure 1. General Location and Features of Alternative X

1.4.3 Project Components Requiring Geotechnical Information and Recommendations

On the uphill (east) side of the highway, existing walls would be removed and a single continuous anchored soldier pile wall with timber lagging, approximately 5,540 feet long and up to 50 feet tall, would be installed. It is anticipated that up to an approximately 400-foot-long section of wall would be tiered to accommodate the road realignment and to improve slope stability and resilience at this location.

On the downhill (west) side of the highway, a single, anchored soldier pile wall with timber lagging, approximately 200 feet long and 15 feet tall, would be installed in a gap between existing walls.

An underground drainage system would be constructed to improve the global stability of the landslide area. The system would consist of three 12-foot diameter drainage gallery tunnels, installed at various elevations subparallel to the slope's contours, with radial drains drilled upward into the slide mass, three interconnected 30-foot diameter vertical shafts, and an outfall structure. There are no roadway tunnels or bridges within this alternative.

1.4.3.1 Proposed Retaining Wall Structures

The proposed Alternative X retaining wall structures are presented in Table 1. The walls would be steel soldier pile walls with timber lagging and ground anchors. The downhill wall is intended to support the roadway prism and prevent sudden loss of the roadway. The uphill walls are intended to accommodate roadway widening while minimizing the project footprint.

Table 1. Alternative X Retaining Walls

Wall No.	Wall Type	Begin Station	End Station	Length (feet)	Maximum Height (feet)	Side
RW 6	Soldier Pile w/ Ground Anchor	479+00	481+00	200	15	Lt
RW 7A-1	Soldier Pile w/ Ground Anchor	455+00	468+65	1,365	50	Rt
RW 7A-2	Soldier Pile w/ Ground Anchor	470+50	479+55	905	50	Rt
RW 7A-3	Soldier Pile w/ Ground Anchor (Terraced)	480+30	513+00	3,270	50	Rt
RW 7B	Soldier Pile w/ Ground Anchor (Terraced)	499+52	503+51	399	50	Rt
RW 7C	Soldier Pile w/ Ground Anchor (Terraced)	499+77	503+02	325	35	Rt

RW 6 would be a fill wall along the southbound side of the highway between the north limit of Existing Wall C (Wilson Creek Wall #83, a solider pile wall with ground anchors installed in 1991) and the south limit of Existing Wall D (South LCG Wall, a soldier pile wall with timber lagging installed in 2015).

RW 7A would consist of a cut wall along the northbound side of the highway.

RW 7B and RW 7C would consist of two separate wall structures, sufficiently offset from one another and from a portion of RW 7A to create flat (terraced) areas between the walls.

1.4.3.2 Proposed Earthwork

Earthwork consists of cuts and fills on roadway tangents and curves and cuts for the proposed anchored soldier pile walls.

1.4.3.3 Proposed Underground Drainage System for Landslide Mitigation

The layout of the proposed Alternative X underground drainage system is shown in Plates 2a and 2b. The underground drainage system is intended to reduce groundwater levels within the hillside that incorporates U.S. 101, thereby reducing pore pressures and increasing the effective stresses acting on the various sliding surfaces and improving slope stability. The underground drainage system would consist of three drainage gallery tunnels with radial drains drilled upward into the slide mass, three interconnected vertical shafts, and an outfall structure.

The drainage gallery tunnels would have outside diameters of approximately 12 feet, and their lengths would range from 6,700 to 7,200 feet. To maximize draining of the slide mass, the drainage gallery tunnels would need to be located below the landslide basal failure zones.

The drainage galleries would be sloped typically 1 to 4 percent towards the vertical shafts to ensure passive gravity flow. The tunnels would be constructed by Tunnel Boring Machines (TBMs) and lined with reinforced precast concrete segments. Small-diameter perforated-pipe drains would radiate outward from the tunnels into the surrounding substrate to capture groundwater and drain the slide mass.

The three vertical shafts would have inside diameters of 30 feet, and their depths would range from 210 to 240 feet. The shafts would be interconnected with 24-inch diameter drain bores leading to the lowest shaft, which in turn would have a single 48-inch diameter bore leading to an outfall structure draining on to a rip-rap slope leading to the Pacific Ocean.

A new 32-foot-wide permanent access road would be provided for construction and maintenance of the underground drainage system (Plate 2a). In addition, a 12-foot-wide temporary access road would be used for construction of the outfall.

1.5 Project Description – Alternative F

1.5.1 General Description

Alternative F would involve constructing a 6,000-foot-long (1.1-mile) tunnel east of the existing highway to avoid the most intense areas of known landslides and instability. Main components would include a tunnel, associated north and south portals and approaches, a bridge from the north portal to connect to existing U.S. 101, and an Operations and Maintenance Center (OMC).

1.5.2 Alignment and Component Locations

Figure 2 is a schematic representation of the general location and features of Alternative F. The alignment is shown in detail in Plates 3a and 3b, and the locations of the Alternative F components are presented in Table 2.

From the south, Alternative F would diverge from the existing highway near the end of the existing truck climbing lane (PM 14.2), traveling approximately 800 feet through a retained excavation and then a 500-foot-long cut-and-cover South Portal structure starting at PM 14.74. The portal structure would open into a two-lane, single-bore tunnel which would be approximately 200 feet below the ground surface (BGS) for most of its length. The tunnel would exit the hillside at the North Portal, near PM 15.6, and the alignment would continue through a retained excavation to the 122-foot-long Wilson Creek Tributary Bridge, a two-lane highway bridge. The alignment would rejoin existing U.S. 101 at PM 15.7. An OMC would be built south of the tunnel to support tunnel operations and maintenance.

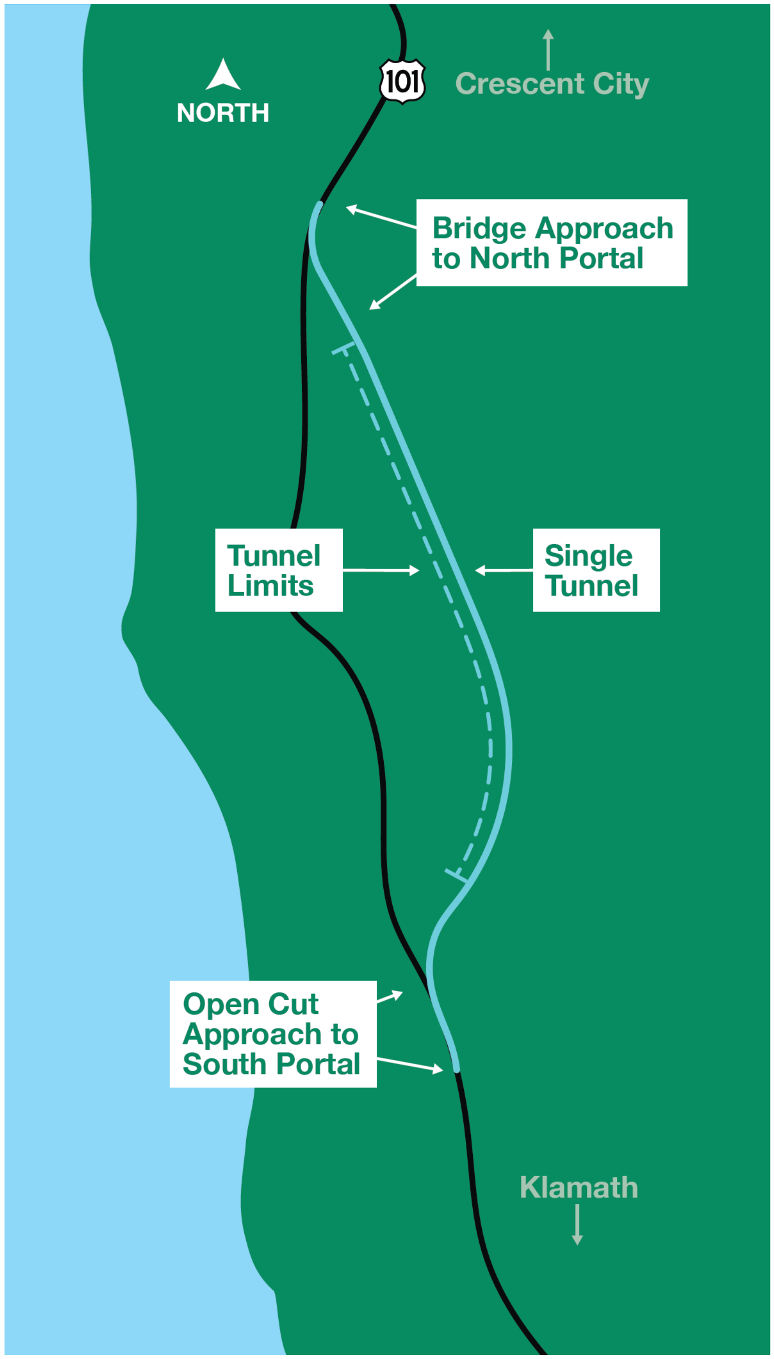


Figure 2. General Location and Features of Alternative F

Table 2. Alternative F Structures

Proposed Structures [1]	Begin Station [2]	End Station [2]	Structure Length (feet)	Total Length (feet)
Roadway at grade	34+36	45+00	1,064	1,064
South Portal Approach	45+00	53+00	800	800
• RW 1 Fill Wall Section (on downhill side)	45+00	49+00	400	
• South Portal Approach, cut section	49+00	53+00	400	
South Portal Cut-and-Cover Section, with EDAS	53+00	58+00	500	500
• RW 2R	52+00	58+00	600	
• RW 2L	53+00	58+00	500	
SEM Tunnel	58+00	116+73	5,873	5,873
North Portal Approach	116+73	119+25	252	252
• RW 3R	116+73	119+48	275	
• RW 3L	116+73	117+43	70	
Wilson Creek Tributary Bridge	119+25	120+47	122	122
Roadway at grade	120+47	127+64	717	717
OMC	~”FS” 33+50	~”FS” 35+00	~150	~150

Note:

(1) Proposed structures, alignment, and stationing are current as of October 26, 2023.

(2) Stationing is along the “F” alignment, unless noted otherwise.

1.5.3 Project Components Requiring Geotechnical Information and Recommendations

1.5.3.1 Proposed South Portal and Approach with EDAS

Near where Alternative F diverges from existing U.S. 101, a concrete retaining wall on spread footings (RW 1) would be constructed on the downhill side (west) of the new road segment. This wall would be up to 20 feet high.

The approach to the South Portal would require a cut-and-cover excavation into the hillside. Retaining walls would be up to 75 feet high, with an average height of 30 feet (RW 2R/2L). The South Portal approach structure would use large diameter secant piles and engineered deformation absorption columns. This Engineered Deformation Absorption System (EDAS) is intended to absorb earthflow movement by using columns engineered to compress over time. As the earthflow continues to move downhill toward the Pacific Ocean, the portal would remain intact.

Once constructed, an intermediate-level slab and a concrete roof structure would be installed over the highway for a length of approximately 600 feet. Soil would be placed on roof and graded to match the surrounding topography. The area would then be revegetated.

1.5.3.2 Proposed Tunnel

The tunnel would be configured for two-way traffic and would be approximately 6,000 feet long. It would be sized to provide truck-height clearance (16 feet, 6 inches) for two 12-foot-wide travel lanes and two 10-foot-wide shoulders. There would be two emergency corridors on either side, and the roofs of these corridors would be bike lanes. The tunnel’s interior spring line width would be, 66.25 feet, and the floor to ceiling height would be 35 feet.

The tunnel would be constructed by the Sequential Excavation Method (SEM), in which the tunnel cross section is subdivided into smaller headings which are excavated and supported sequentially. The initial lining for the SEM tunnel would consist of flashcrete, rock bolts, lattice girders, and shotcrete. The flashcrete is intended to provide temporary cohesion for the exposed tunnel walls as the area is mucked and rock bolts are installed. The initial lining support would be in service for approximately one year.

The final lining would consist of cast-in-place (CIP) reinforced concrete. The tunnel lining design would incorporate a full-round permanent waterproof membrane to prevent groundwater inflow.

The tunnel would have an invert drain to collect any water that might be generated by vehicles or leakage. The tunnel profile would slope downward toward the south, and tunnel drainage would be directed to a holding facility near the South Portal for disposal.

The tunnel would include various safety features, including ventilation, lighting, longitudinal pressurized chambers for emergency egress, emergency communications systems, equipment chambers, and a fire suppression system.

1.5.3.3 Proposed North Portal and Bridge Approach

The tunnel would exit the hillside north of the existing slide. The North Portal headwall and immediate rock slopes would be supported by permanent rock bolts and CIP facias, while the portal approach would be supported by retaining walls (RW 3R/3L) which are anticipated to be cast-in-drilled-hole (CIDH) piles and lagging with permanent ground anchors. These retaining walls would be up to 30 feet high and would be at the south end of the Wilson Creek Tributary Bridge connecting the portal headwall to U.S. 101.

The Wilson Creek Tributary Bridge at the north portal location would be a single-span, pre-cast, concrete girder bridge approximately 122 feet long and 48 feet wide, with a single 12-foot-wide lane in each direction and 10-foot-wide shoulders. A new culvert would be installed under the northern tunnel approach between the bridge and the northern portal. The culvert would be 24 inches in diameter or larger, and approximately 200 feet long.

1.5.3.4 Proposed Operations and Maintenance Center (OMC)

The OMC would be located south of the tunnel at PM 13.52, and would include a building, parking spaces, and outdoor storage, as well as maintenance, operations, and emergency equipment. The building would be an approximately 12-foot-tall, 18,000-square-foot, single-story structure. The structure would be founded on rigid shallow foundations bearing on colluvium.

Retaining walls with perimeter chain link fencing would be located around the OMC building and yard for security purposes and to provide a grade break that allows the OMC facilities to be placed below the existing ground surface. The site grading will be achieved using cut slopes with perimeter chain link fencing for security.

2 GEOTECHNICAL INVESTIGATION

2.1 Summary of Geotechnical Investigations

To date, three phases of geotechnical investigations have been performed for the project, which were identified as Phase 1, Phase 2A, and Phase 2B. Details of work performed for each of these phases is provided below.

The Phase 1 geotechnical investigation program was completed between February 5, 2018 and September 27, 2018. Investigation work performed for this program included the following:

- Literature review of existing reports, published geologic literature, and maps to be incorporated into a site geologic model.
- Review of historic aerial photographs to evaluate land use practices and past slope instability. High resolution topographic maps, as well as hill-shade and percent slope images, were generated using available Light Detection and Ranging (LiDAR) data sets. These were examined, and the landforms interpreted to assess potential slope instability features. To improve the efficiency of field mapping, specific areas were identified during review as target areas for further evaluation.
- Reconnaissance-level field review of target areas identified during the desktop study, as well as walking, and driving much of the network of roads and skid-trails crossing the study area. Field reconnaissance to map seepage patterns as well as the distribution of different soil and bedrock materials exposed in natural outcrops and road cut exposures, and then evaluation of their impact on slope stability. Field-verification of desktop mapped features, mapped recent slope failures, evaluated landforms, and observed tilting and bowing of

trees.

- Thirteen borings advanced at eight locations. Each boring location was chosen to provide information needed to evaluate the proposed alignment or to confirm the presence or activity of landslides identified in previous investigations.
- Stand-pipe monitoring wells constructed in eight of the 13 borings, and vibrating wire piezometers (VWPs) and data loggers installed in these wells to continuously monitor changes in groundwater levels.
- Slope inclinometers (SIs) installed in five of the 13 borings to measure ground displacement in suspected landslide features.
- A series of eight seismic refraction surveys conducted at key locations to image subsurface structures such as landslides; aid in the lateral correlation of geotechnical borings; and provide data to aid the evaluation of engineering characteristics of rock and soil along the alignment.

The Phase 2A geotechnical investigation program was completed between February 5, 2018 and September 27, 2018. Investigation work performed for this program included the following:

- Seven geotechnical borings (five vertical and two horizontal) advanced and logged in conformance with Caltrans (2010) Soil and Rock Logging, Classification, and Presentation Manual.
- SI casing installed in each geotechnical boring for slope monitoring.
- Two VWP borings drilled using air rotary methods. No geotechnical information other than groundwater data were acquired from VWPs.

The Phase 2B geotechnical investigation program included field reconnaissance mapping by geologists from Caltrans, Kleinfelder, and SHN on May 4 through May 6, 2020 and field exploration work September 22 through January 14, 2021. The Phase 2B program included the following:

- Desktop study using available LiDAR elevation data, aerial photographs, and review of historical reports and data to develop an exploration plan and to prepare a geologic hazards map.
- Geologic mapping to further collect field data related to features associated with landslide activity within the project area, and to further refine the geologic hazards map completed during the desktop study.
- Subsurface explorations to obtain rock and soil samples at strategic locations relative to alignment alternatives and identified geologic hazards.
- Logging of borings using the Caltrans (2010) Soil and Rock Logging, Classification, and Presentation Manual, and collecting, handling, labeling, and storage of core and soil sample specimens in core boxes in preparation for further classification and future selection of samples for laboratory testing.

- Downhole acoustic televiewer (ATV) and optical televiewer (OTV) surveys, downhole geophysics using P- and S-wave suspension logging for shear and primary wave velocity data, and downhole pressuremeter testing for assessment of states of stress and strain/stiffness characteristics of various rock formations at variable depths.
- Packer testing at select intervals and boreholes to obtain hydrogeologic data.
- VWPs and open-hole standpipe type wells installed to collect groundwater data.
- SI casing and time domain reflectometry (TDR) cable installed to collect deformation data associated with ground movement.
- Surface geophysical surveys to further characterize subsurface conditions and to obtain information on rippability for earthwork grading.
- Weather stations installed to measure rainfall.
- Laboratory testing of rock and soil units for characterization of material physical and engineering properties, mineralogy, shear strength, permeability, suitability for reuse in earthwork, compaction characteristics, and slope stability analysis.

2.2 Preliminary Geotechnical Data Report

The Final Preliminary Geotechnical Data Report (Caltrans, 2022a) was issued in July 2022 and presents geotechnical data gathered by or on behalf of Caltrans for previous LCG Grade Project studies (Investigation Phase 1 and Phase 2A), subsequent geologic and geotechnical data gathered by and on behalf of Caltrans through May 31, 2021 (Phase 2B), as well as published reports relevant to the Project area. Data collected by Caltrans prior to Phase 1 was not included in the data report, as directed by Caltrans.

3 GEOTECHNICAL CONDITIONS

3.1 Geology

3.1.1 Regional Geology and Seismicity

The project area is located within the Coast Ranges geomorphic province of California, near the Klamath Mountains which lie about 10 miles to the east. The site is located about 90 miles north of the Mendocino Triple Junction, which is the crustal intersection of the Pacific, North American, and Gorda/Juan de Fuca tectonic plates. North of the triple junction, the Gorda/Juan de Fuca plate is being subducted eastward beneath the North America plate along the Cascadia Subduction Zone (CSZ), which extends approximately 800 miles from northern California to Vancouver Island, British Columbia. As is true for other coastal regions of northern California, Oregon, and Washington, the project site overlies the interface associated with the subducting crustal plate. This subduction interface is a low angle, east-dipping “megathrust” fault capable of generating great earthquakes.

The site geologic setting is characterized as being within the accretionary prism that has formed (and continues to form) above the CSZ at the leading edge of the North America plate. Geologic materials in the region are primarily associated with the long-term accretionary history, and active tectonic deformation throughout the region occurs as a byproduct of the ongoing subduction process. In addition to the immense seismic potential associated with the CSZ itself, other active seismic sources also occur within the subducting Gorda plate and along secondary faults associated with fold and thrust belts within the over-riding North American plate.

The Coast Ranges in the project area are underlain by regionally extensive Mesozoic and Cenozoic age rocks of the Franciscan Complex, an assemblage of mostly marine sedimentary materials accreted ("welded" or "scraped off") to the continental margin. The Franciscan Complex occurs regionally in a series of elongate belts that define specific age materials, material types, and metamorphic grades; these are the Coastal, Central, and Eastern belts. The site occurs within the Eastern belt of the Franciscan Complex (Delattre and Rosinski, 2012, Aalto, 1989), which is the oldest, least sheared, and most highly metamorphosed of the three belts (McLaughlin et al., 2000). In the project area, the Franciscan Complex is bound on its east side by the Coast Range fault, separating it from older Klamath Mountain rocks to the east.

3.1.2 Area Geology

The Franciscan Complex in the site vicinity consists of two primary units: argillite-matrix Melange and a variety of "broken formation" units that originated as submarine debris slide deposits (e.g., "turbidites") that consist mostly of interbedded sandstone and shale beds. The Melange in this case is interpreted as a large submarine landslide deposit (olistostrome) that is in depositional contact with the underlying turbidite ("broken formation") sequence (Aalto, 1989). Subsequent extensive accretion-related deformation (faulting, metamorphism) has resulted in pervasive shearing and complex structural relationships within the two primary bedrock types.

Bedrock mapping of the project area has been done by Ristau (1979), Aalto and Harper (1982), Wills (2000), and Delattre and Rosinski (2012). These maps generally show the project area crossed by an elongate north-trending band of Melange (unit KJFm) surrounded by the "Broken Formation" (unit KJFbf), as shown on Wills (2000, Plate 5). On the map by Delattre and Rosinski (2012), included as Plate 4, the Melange belt is referred to as the "Melange of the Crescent City area" and is shown extending from Point St. George in Crescent City to the Lost Man fault south of the Klamath River mouth. Importantly, the contact bounding the "Melange of the Crescent City area" is shown as "gradational," reflecting its interpretation by others as depositional.

As such, bedrock in the project area can be characterized by sheared turbidites (Broken Formation) that transition eastward to a chaotic olistostrome deposit (Melange) where a variety of rock types may occur. The contact between these two units appears to intersect the coastline within the project area, as Broken Formation materials are exposed in the northern part of the project area and Melange bedrock in the south. The location of the contact between Melange and Broken Formation extends northward through the project area.

Broken Formation rocks in the project area consist mainly of thickly bedded, gray sandstone with lesser siltstone and shale interbeds. The material occurs as relatively intact blocks of varying sizes bounded by shear zones; therefore, bedding is discontinuous. Due to the preponderance of sandstone, Broken Formation areas are relatively resistant to erosion such that drainages are well-defined and more mature topographic (and forest) conditions develop. The sandstone component of the Broken Formation is characteristically jointed, forming a blocky, “broken” texture with a general lack of cohesion between individual blocks.

Because Melanges in the project area are interpreted as having been derived in a large submarine landslide (olistostrome), they consist of isolated, rootless rock blocks entrained within a highly sheared, dark gray siltstone or argillite matrix. The material appears to exhibit “block-in-matrix” texture typical of Melanges elsewhere in the Franciscan Complex. Rock blocks (“olistoliths”) vary in size, lithology and location; larger blocks are mappable in scale (Wills, 2000, Plate 5). Due to the weak nature of the sheared Melange matrix, these areas have a high susceptibility to earthflows and erosion and form a distinct hummocky, low gradient topography.

3.1.3 Area Geologic Hazards

The LCG segment of U.S. 101 traverses a steep coastal slope underlain by highly to pervasively sheared bedrock and is subject to high levels of rainfall and seismicity. It is subject to a variety of very active landslide processes. These range from shallow debris slides and rockfalls, deep to very deep translational landslides (some of which may toe out at or below the shoreline), and earthflows.

Landsliding inland of the coastal slope is less dynamic and the majority of the slope failures appear to be dormant; generally initiating in pre-historic time.

3.1.3.1 Landslides

- *Landslide Complexes*

The entire coastal slope along the LCG segment is unstable and underlain by several landslide “complexes” or actively failing areas, with multiple failure planes at varied depths. Based on geologic and geomorphic distinctions, however, the slope can be differentiated into four individual landslide complexes. A landslide classification chart, which describes the mapping criteria for this study is presented on Plate 6, and the landslides complexes are shown on Plate 7. From north to south, they are referred to as the North Last Chance Grade Complex (NLCG), South Last Chance Grade Complex (SLCG), Wilson Creek Complex (WC), and the Large Earthflow Complex (EF).

The NLCG, SLCG, and WC are underlain by Broken Formation bedrock and occur in the steepest, tallest part of the subject coastal slope. These deep-seated complexes occur as nested slides developing above a deep basal slide surface that likely daylights at or near the shoreline.

The shallowest sliding in the three northern slide complexes occurs as debris slides and rockfalls on over-steepened slopes on the bluff face where coastal erosion is occurring, or on/above cut slopes along the highway. The highway through this area essentially follows a break-in-slope between the steep coastal bluff face and the moderate-gradient forested slope above the highway that leads to the coast-parallel ridge line. The slope below the highway is, in many places, a bare cliff with coalescing debris slides that may extend hundreds of feet up the cliff face; many of these reach the outboard edge of the highway corridor. Over time, the coalescing debris slides form deep coastal ravines, further developed by surface erosion, leading to the shoreline. Individual debris slides are usually less than a few tens of feet deep or wide and are subject to rapid failure, especially when high levels of moisture are present, after periods of sustained high surf, rain or during strong seismic shaking.

Rockfalls are common in these areas due to the distinctly jointed, "broken" nature of the underlying bedrock. Rockfalls occur on cut slopes along the highway, sometimes advancing upslope and expanding into translational failures. Rockfall failures may occur at any time of the year, but generally occur more often during periods of sustained heavy rainfall or following seismic shaking events.

Deeper landsliding in the NLCG, SLCG, and WC appears to occur as a series of nested translational or locally trans-rotational slides, with a deep basal failure surface/zone that in places exceeds 200 to 300 feet in depth. This basal surface/zone likely daylights at or near the modern shoreline. The upper limit of the deep-seated sliding extends all the way to the ridge top above the highway, where a complex series of distinct, interconnected, geomorphically young head scarps are apparent. A portion of the NLCG head scarp appears to extend immediately above U.S. 101 near the north end of both Alternative X (Station 512+00 to 518+10) and Alternative F (Station 115+00 to 120+00). This head scarp, along with the geomorphology below U.S. 101 and subsurface conditions observed in borings suggest portions of the slide mass may be nearly evacuated in this area of the highway.

Distinct internal scarps within the body of the slides are present as well, appearing to define individual slide bodies or movement centers. The deeper sliding is interpreted as translational because old-growth redwood trees (hundreds of feet tall, hundreds of years old) on the slope above the highway, while notably tilted and warped near the edges of the individual blocks, are relatively intact and un-tilted in the intervening areas despite large amounts of documented lateral movement, as well as from subsurface data collected to date. Available SI data shows multiple active failure surfaces/zones within the bodies of the slides, documenting complex slide movement at depth.

The EF occurs at the southern end of the LCG segment of U.S. 101, in an area underlain by Melange bedrock. The contact between Broken Formation and Melange along the highway corridor occurs at about PM 14.45. South of this contact, which is interpreted to dip toward the south, the entire coastal slope is affected by the EF. The earthflow is characterized by subdued hummocky topography, an absence of well-defined drainage channels, and distinct vegetation changes (no old growth trees). The EF extends to the ridgeline, where a large sandstone block within the Melange occurs as shown on Plate 5. The earthflow is associated with a relatively continuous, moderate gradient slope (generally ranges from 15 to 30 percent from its head at the ridgeline all the way to the shoreline. Where the earthflow reaches the shoreline, the slope transitions to a bluff and there is an increase in offshore rock blocks ("sea stacks"), reflecting past erosion of the Melange within the earthflow.

The inland portion of the study area is forested ground that is associated with slope gradients that are generally less steep than the coastal slope. In this area, landslides are interpreted as mostly dormant, deep-seated translational landslides and some earthflows. In these areas, landslide morphology is muted and generally indistinct, and the ground is capable of supporting undeformed old-growth forests that, in this environment, can be several hundred to a couple of thousand years old. On the Landslide Map most slides in the inland part of the project area are classified as either “Dormant” or “Ancient”. Many of the slides are interpreted as covering large areas encompassing entire slopes or sub-basins. The scale and apparent dormancy of these landslides suggests they are the result of long-term events or cycles that pre-date or exceed the historical record (great earthquakes, long-term geomorphic response to sea level change, for example).

- *Landslide Activity*

The discussion of landslide activity at LCG is best described in two parts, one focused on the coastal slope and one focused on inland slopes.

Landslides on inland slopes are interpreted as largely being “dormant” or “ancient” in age. Interpretation of landslide age in these areas is largely based on geomorphic interpretation, which suggests considerable antiquity. In general, there has been significant landscape modification (erosion, drainage development, vegetation regrowth) in these areas since most of these slides were active. The presence of old growth redwood trees on the slopes (currently within parks and previously on adjacent industrial timberlands) suggests a relatively stable landscape. SIs in these areas exhibit no movement (although the available measurement record is relatively short).

Landslides on the coastal slope are uniformly interpreted as “active” features on the Landslide Map. Geologic and geomorphic conditions, the continuous record of maintenance and repair since the inception of the highway, repetitive monument surveys, comparisons of two sets of LiDAR imagery, and the results of ongoing SI monitoring, all indicate the landslides on the coastal slope are in a semi-continuous state of activity.

Preliminary comparisons of 2011, 2016, and 2020 LiDAR imagery (Cambio, BGC Engineering, Inc., 2021) show the highest rates of ground level changes occurring on lower bluff slopes subject to frequent, relatively shallow debris sliding that is likely due to coastal erosion. Deeper slides appear to move episodically, or at relatively slow rates, until likely triggered by large seismic or weather events. Movement of deep bedrock slides has impacted significant sections of the highway periodically throughout the history of LCG, requiring substantial roadway repairs that continue today at relatively slow rates until likely triggered by large seismic or weather events.

SI data on the coastal slope has come primarily from geotechnical borings along the highway corridor, supplemented by recent borings on the forested slope above the highway. Available inclinometer data suggests ongoing movement across much of the coastal slope, although at widely varying rates. Several inclinometers indicate movement on multiple slide planes within an individual slide mass within the same measurement interval. Inclinometer casings in more dynamic parts of the slope typically last only a year or two before becoming sheared off/unusable (requiring several inches of displacement).

A series of grabens at the crown of the NLCG, SLCG, and WC appear to record a large, older slide event. This event is tentatively correlated with the most recent CSZ earthquake in 1700, although additional work would help support this interpretation. If this is a valid interpretation, the graben would reflect a large coseismic landslide event that would have encompassed much of the coastal slope.

3.1.3.2 Surface and Subsurface Movements

Landslide movement and the interpretation of potential failure surfaces is primarily derived from SI data and slope geometry. SI data presented in the Final Preliminary Geotechnical Data Report (Caltrans, 2022a) and cross-sectional analysis suggest movement along basal failure surfaces/zones approaching depths of approximately 80 to 100 feet within the NLCG, approximately 275 feet within the SLCG, and approximately 276 feet within the WC. There is 2012 data indicating NLCG movement at a depth of 160 feet. As nested landslide complexes, inclinometers in these areas show movement across a wide range of depths, sometimes on multiple slide planes within the same slide mass and within the same measurement interval.

Inclinometer data and cross-sectional analyses within the EF indicate movement along non-uniform, basal failure surfaces/zones varying from 45 feet to as deep as 144 feet.

3.1.3.3 Coastal Erosion

The coastline along the LCG segment is characterized by a narrow beach strand with few offshore rocks to buffer the base of the cliff from large ocean waves that occur with regularity during the winter months. In this environment, coastal erosion is intense and a ubiquitous part of the winter season. Aerial photographs of the coastline are frequently marked by a substantial sediment plume occurring offshore of the LCG.

Future coastal erosion rates will be impacted and expected to increase by projected sea level rise along most of the California coast. However, according to the California Office of Environmental Health Hazard Assessment (2023), the Crescent City coastline is the exception as sea level is dropping (3 inches lower than in 1933). Local sea level measurements using tide gauges reflects the combined effects of regional sea level rise and vertical land movement. In the Crescent City area, tectonic uplift is outpacing sea level rise and, therefore, the relative effect is the lowering of sea level.

3.1.3.4 Rockfalls

Based on Caltrans maintenance records and evidence of multiple rockfalls that occurred during Phase 2B investigations, rockfall occurs locally in Broken Formation bedrock along the LCG corridor. This rockfall is likely the result of loose, blocky rock exposed at the ground surface on steep slopes and was observed following periods of heavy rainfall.

3.1.3.5 Seismicity

The site area and environs are characterized as a region of high seismic potential. A map of regional faults and recent earthquake epicenters is included as Plate 8. The area south of the project study area near the Mendocino triple junction is perhaps the most seismically active area in the conterminous United States (Freymueller et al., 1999; Furlong and Schwartz, 2004; Dengler, 2008). The CSZ is capable of generating “great” earthquakes of high magnitude (>M8.5), depending on the length of the rupture (Heaton and Hartzell, 1987; Nelson and others, 2021; PNSN, 2020). A full-length rupture of the entire CSZ would likely exceed magnitude M9. The surface trace of the CSZ is located about 55 miles west of the site (measured from Google Earth Pro), while the fault plane dips eastward about 10 to 15 degrees (McLaughlin et al., 2000) beneath the region. The CSZ detachment fault boundary is therefore located about 9½ to 14 miles deep beneath the site.

Recent estimates from stratigraphic records suggest 17 major earthquakes have occurred along the southern and central segments of the CSZ in the past 6,700 years, with earthquake recurrence on the order of 510 to 540 years (Nelson et al., 2021). The most recent major CSZ earthquake occurred on January 27, 1700, and is interpreted as a >M9 full-length CSZ rupture. That earthquake is documented in local native tribal oral history, Japanese historical tsunami records and is documented in the field by land level elevation changes from California to British Columbia (Atwater et al., 2005).

The Gorda plate is a relatively small tectonic plate at the southern end of the CSZ and it is subject to a variety of complex forces as it is being subducted. It is actively deforming and is the most frequent source of felt earthquakes for the northern California coast area (Chaytor et al., 2004; Hemphill-Haley et al., 2020). Due to the internal stresses within the Gorda plate, it is highly sheared, but most notably broken by a series of northeast-trending faults that produce frequent earthquakes along left-lateral faults. Faulting within the Gorda plate produced 20 earthquakes >M5.9, including four >M7 earthquakes, between 1976 and 2010 (Rollins and Stein, 2010). There have been three additional earthquakes >M6.5 since 2010.

Active deformation is occurring in a fold-and-thrust belt terrain south of the project area, responding to northeast-southwest oriented crustal shortening. There is a series of northwest-trending, southwest-vergent (over-riding block moving toward the southwest) thrust faults which include the Mad River fault zone, Table Bluff fault and Little Salmon fault. These faults are located between 37 and 60 miles south of the project area measured along the coastline; although all are known to extend offshore (Clarke, 1992; Clarke and Carver, 1992; Hemphill-Haley et al., 2020). These likely represent the nearest known Holocene-active surface faults to the project site.

A series of suspected older, poorly defined bedrock faults occur north of Big Lagoon and generally south of the Klamath River, located 4½ to 10 miles to the south and southeast of the project area. These include the Bald Mountain-Big Lagoon fault zone, Grogan fault, Lost Man fault and Surpur Creek fault. These faults are considered to be "Quaternary" age (Bald Mountain-Big Lagoon fault is listed as "late Quaternary" age) by the U.S. Geological Survey (USGS) Quaternary Fault and Fold Database. The Grogan fault defines a major geologic bedrock boundary and is interpreted as a high-angle right-lateral strike-slip fault (Hart, 1999; Kelsey and Carver, 1988). The Lost Man and Surpur Creek faults (generally defined by the mapping of Aalto et al., 1982) are poorly located, but represent the nearest mapped faults to the project area. An early map (Aalto et al., 1982) shows a northward extension of the Lost Man fault that crosses the project area; this trace appears on an outdated California Geological Survey (CGS) Fault Activity Map (until 2017). The northern extension of the Lost Man fault is not shown north of the Klamath River on more recent mapping (it extends offshore; Kelsey and Carver, 1988) and is not shown on the current national database of Quaternary faults and folds (Bryant, 2017). Evidence for this fault was not observed in the field during previous LCG-related geologic investigations.

3.1.4 Area Geologic Map

Geologic and landslide mapping was initially developed during a desktop study that included review and use of the following data:

- Published geologic maps within the site vicinity
- Caltrans and consultant unpublished reports for LCG, including the previous 2018 Expert-Based Risk Assessment report by BGC Engineering USA Inc.
- Caltrans and consultant borings (logs) drilled within the existing right-of-way for various previous projects along LCG as well as selected off-site explorations on the Green Diamond Property
- Caltrans and consultant inclinometer plots along LCG
- Prior Caltrans mapping of the Project area
- Elevation and aerial photography including 2016 based LiDAR, Google Earth Imagery, Digital Elevation Models (DEMs), and ortho aerial imagery

Landslide features were initially mapped from topographic variations in elevation relief viewed on LiDAR and DEM and features viewed in aerial photograph imagery. The desktop mapped area included the approximate area bound to the north by the Project limits at PM 16.0, to the west by the ocean/beach boundary and to the south by the Project limits near PM 12.0 and extending northeast along Wilson Creek Road to the east boundary. The east boundary was less defined as it included the eastern most U.S. 101 alignment segments and adjacent slopes.

To characterize the existing landslides/slope instabilities the geologic team used a four-digit classification system. Each of the digits represents a specific classification characteristic that describes the type of landslide. The four characteristics are State of Activity, Certainty of Identification, Dominant Type of Movement, and estimated Thickness of Deposit. A description of the landslide characteristics is presented on the Landslide Identification Chart and Geologic Units Descriptions, Plate 6.

In general, the landslides within the project boundaries were classified as either Active, Dormant, or Ancient. Active landslide features, defined by distinct landslide head scarps and side scarps and hummocky, uneven topography, were mapped with high confidence. Less distinct topography and elevation relief were more commonly associated with dormant and ancient landslide features, primarily found to the east of the primary ridge.

Field reconnaissance mapping was performed by geologists from Caltrans, Kleinfelder, and SHN on May 4 through 6, 2020. Two teams of four covered the Project area to evaluate landslide hazards and assess larger features mapped from the desktop study. The field reconnaissance study area included the area bound to the north by the Project limits at PM 16.0, to the west by U.S. 101 and to the south by the Project limits near PM 12.0 and extending northeast along Wilson Creek Road to the east boundary. Similar to the desktop study the east boundary was less defined and included only larger landslides that were identified as directly impacting a conceptual project alignment. Not all landslides and/or portions of landslides could be mapped or verified during the site reconnaissance due to heavy vegetation that prohibited access. During the field reconnaissance, larger, ancient landslides were interpreted from topographic gradient changes and observed accumulation and distribution of colluvial soils. Smaller, recent landslides were interpreted from evacuated zones with an abnormal or absent sediment accumulation at the base of slopes and drainages. Tilting and bowing of large redwoods were also used in the interpretation of relative landslide movement and age of initiation. The desktop map was subsequently adjusted to reflect observations made during the reconnaissance. Lastly, mapping from both the desktop and reconnaissance has been updated and applied using the 2020 LiDAR hillshade/topographic base, and is presented as Plate 7, Project Landslide Map.

3.1.5 Geologic Cross Sections and Profile

Cross sections were constructed through each of the four landslide complexes on the west side of the LCG ridgeline. The sections were positioned in representative locations within each complex in close proximity to existing boring and inclinometer data and were oriented approximately perpendicular to contour (parallel to the direction of apparent landslide movement). The sections extend east from the shoreline across the primary ridge and transect Alternatives X and F. Geologic contacts were applied to the sections utilizing projected boring data, inclinometer data and the mapping presented on the Project Landslide Map (Plate 7). Geologic Cross Sections 1 through 4 are presented on Plates 9a through 9d.

3.1.5.1 Alternative X

The proposed Alternative X alignment essentially parallels the existing roadway alignment, transects the NLCG, SLCG, and WC landslide complexes, and extends approximately 530 feet into the north portion of the EF. As such, Alternative X would be underlain by landslide deposits at depths well below the proposed construction. The drainage system shafts and the portions of the three drainage gallery tunnels located within the EF are south of Cross Section 1, but the other portions of the drainage gallery tunnels are shown on Cross Sections 2 through 4. Cross Sections 2 through 4 also show the position of the U.S. 101 within the WC, SLCG, and NLCG, with the basal failure surface located at depths of approximately 240 feet, 250 feet, and 74 feet, respectively.

3.1.5.2 Alternative F

Geologic contacts were applied to the Alternative F profile utilizing intersection points with Geologic Cross Sections 1 through 4 (Plates 9a through 9d) along with the mapping presented on the Project Landslide Map (Plate 7). The proposed Alternative F alignment is underlain by or is located within surficial colluvium and the EF from its south terminus at Station 34+36, continuing through the South Portal approach. The cut-and-cover section is planned to be excavated through EF and Melange, encountering the inferred basal failure surface of the EF at approximately Station 56+00. Full-face SEM excavation in Franciscan Complex Melange bedrock would begin at Station 58+00.

At approximately Station 67+20, the tunnel alignment would encounter the postulated contact between the Franciscan Complex Melange and the Franciscan Complex Broken Formation bedrock. At this point, the centerline of the alignment would be located approximately 150 feet laterally east of the WC head scarp and approximately 350 feet from the apparent basal failure surface at its closest point. The alignment would remain within the Broken Formation bedrock and east of the landslide complexes up to the North Portal at Station 116+73 where it would exit the ground surface through the Broken Formation and a thin layer of colluvium. The portal would transition to a proposed bridge at Station 119+25 where it would span a colluvial drainage underlain by Broken Formation bedrock to Station 120+47. The proposed alignment would remain underlain by colluvium and the Broken Formation until conforming back to U.S. 101 Station 127+00.

The geologic profile for the Alternative F tunnel is presented on Plate 10.

3.2 Topsoil – Soil Survey Review

According to the United States Department of Agriculture Web Soil Survey (USDA, 2021), the two proposed alignments were mapped by two soil surveys. Most of the project area is found in the Redwood National and State Parks Soil Survey Area (CA Soil Survey #796) and includes Alternative X and both tunnel portals of Alternative F. A small portion of the site is mapped in the Humboldt-Del Norte Soil Survey Area (CA Soil Survey #605).

According to the soil surveys, the project site is underlain by four soil complexes. Alternative F is underlain by Unit 590 (Sasquatch-Yeti-Footstep Complex), Unit 591 (Sasquatch-Sisterrocks-Ladybird Complex), and Unit 592 (Sisterrocks-Sasquatch-Footstep Complex). Alternative X is underlain by Unit 592 (Sisterrocks-Sasquatch-Footstep Complex) and Unit 594 (Sisterrocks-Sasquatch-Houda Complex).

Physical characteristics of each soil component can be found in Table 3. Characteristics include the soil profile, soil description, and erosion factors. Soil erosion factors include Kf (rock-free), Kw (whole soil), and T factors. Erosion factors Kf and Kw indicate the susceptibility of a soil to sheet and rill erosion by water. Factor Kf estimates the erodibility of material less than 2 millimeters in size, and Kw estimates the erodibility of the entire soil unit. Higher values indicate increased rates of soil loss. The T factor is an estimate of the maximum average annual rate of soil erosion by wind and/or water over a sustained period in tons per acre per year. Neither alternative is underlain by a soil complex or component designated on the USDA Highly Erodible Land (HEL) list.

Table 3. Topsoil Physical Characteristics

Map Symbol	Soil Name	Profile	Depth (inches)	Soil Description	Erosion Factors		
					Kw	Kf	T
590	**Sasquatch-Yeti-Footstep Complex**						
	Sasquatch Slope: 5 to 30 percent	Oi	0 to 2	Slightly Decomposed Plant Material	-	-	5
		A	2 to 19	Loam	0.28	0.28	-
		Bt1	19 to 43	Clay Loam	0.24	0.24	-
		Bt2	43 to 65	Clay Loam	0.24	0.24	-
		Bt3	65 to 79	Paragravelly Clay Loam	0.24	0.24	-
	Yeti Slope: 5 to 30 percent	A	0 to 16	Loam	0.32	0.32	5
		Bt1	16 to 37	Clay Loam	0.28	0.28	-
		Bt2	37 to 51	Gravelly Clay	0.15	0.28	-
		C	51 to 60	Gravelly Clay	0.15	0.28	-
	Footstep Slope: 5 to 30 percent	A	0 to 15	Gravelly Loam	0.17	0.28	2
		Bt	15 to 26	Very Gravelly Clay Loam	0.05	0.28	-
		CBt	26 to 31	Extremely Gravelly Clay Loam	0.05	0.37	-
		R	31 to 41	Bedrock	-	-	-
591	**Sasquatch-Sisterrocks-Ladybird Complex**						
	Sasquatch Slope: 30 to 50 percent	Oi	0 to 1	Slightly Decomposed Plant Material	-	-	5
		A	1 to 17	Loam	0.32	0.32	-
		Bt1	17 to 46	Gravelly Clay Loam	0.15	0.24	-
		Bt2	46 to 56	Clay Loam	0.28	0.28	-
		Bt3	56 to 79	Gravelly Clay Loam	0.17	0.32	-
	Sisterrocks Slope: 30 to 50 percent	A1	0 to 9	Loam	0.20	0.20	3
		A2	9 to 16	Gravelly Clay Loam	0.10	0.20	-
		Bt1	16 to 41	Very Gravelly Clay Loam	0.10	0.28	-
		Bt2	41 to 67	Very Gravelly Silty Clay Loam	0.10	0.37	-
	Ladybird Slope: 30 to 50 percent	A	0 to 7	Gravelly Loam	0.10	0.24	5
		AB	7 to 15	Gravelly Silty Clay Loam	0.15	0.28	-
		Bt	15 to 55	Gravelly Clay Loam	0.15	0.24	-
		CBt	55 to 60	Very Gravelly Loam	0.10	0.32	-
592	**Sisterrocks-Sasquatch-Footstep Complex**						
	Sisterrocks Slope: 50 to 75 percent	A	0 to 7	Gravelly Loam	0.15	0.28	2
		BA	7 to 13	Very Gravelly Clay Loam	0.05	0.24	-
		Bt1	13 to 32	Extremely Gravelly Sandy Clay Loam	0.05	0.20	-
		Bt2	32 to 60	Extremely Gravelly Clay Loam	0.02	0.32	-
	Sasquatch Slope: 50 to 75 percent	Oi	0 to 2	Slightly Decomposed Plant Material	-	-	5
		A	2 to 16	Gravelly Loam	0.15	0.28	-
		BAt	16 to 23	Gravelly Clay Loam	0.10	0.24	-
		Bt1	23 to 53	Gravelly Clay Loam	0.15	0.24	-
		Bt2	53 to 60	Very Gravelly Clay Loam	0.15	0.37	-
	Footstep Slope: 50-75 percent	A	0 to 7	Gravelly Loam	0.17	0.32	2
		Bt1	7 to 14	Very Gravelly Loam	0.10	0.32	-
		Bt2	14 to 28	Extremely Gravelly Clay Loam	0.05	0.37	-
		R	28 to 37	Bedrock	-	-	-
594	**Sisterrocks-Sasquatch-Houda Complex**						
	Sisterrocks Slope: 30 to 75 percent	Oi	0 to 1	Slightly Decomposed Plant Material	-	-	5
		A	1 to 8	Gravelly Loam	0.17	0.28	-
		AB	8 to 16	Very Gravelly Loam	0.10	0.28	-
		Bt	16 to 47	Very Gravelly Clay Loam	0.05	0.28	-
		C	47 to 60	Very Gravelly Clay Loam	0.10	0.32	-
	Sasquatch Slope: 30 to 75 percent	Oi	0 to 2	Slightly Decomposed Plant Material	-	-	5
		A	2 to 20	Loam	0.28	0.28	-
		Bt1	20 to 41	Clay Loam	0.28	0.28	-
		Bt2	41 to 79	Gravelly Clay Loam	0.17	0.32	-
	Houda Slope: 30 to 75 percent	Oi	0 to 1	Slightly Decomposed Plant Material	-	-	4
		A	1 to 8	Gravelly Loam	0.17	0.28	-
		BA	8 to 15	Gravelly Clay Loam	0.15	0.28	-
		Bw1	15 to 22	Very Gravelly Clay Loam	0.10	0.32	-
		Bw2	22 to 33	Very Gravelly Clay Loam	0.10	0.32	-
		C1	33 to 53	Very Gravelly Clay Loam	0.10	0.32	-
		C2	53 to 60	Extremely Gravelly Clay Loam	0.05	0.32	-

3.3 Area Groundwater Conditions

3.3.1 Area Groundwater Regime

The project site is located in the North Coast Hydrologic Region (DWR, 2021), which is bounded to the west by the Pacific Ocean, to the north by the Oregon border, to the south by San Francisco Bay, and to the east by the Klamath Mountains and Central Valley hydrogeologic provinces. Specifically, the project site is located within the coastal basins area of these hydrogeologic provinces (Belitz et al., 2003; Mathany et al., 2011). According to Caltrans (2020e and 2021a) the site is within Cal Water watershed North Coast Hydrologic Region's Smith River Hydrologic Unit and Klamath River Hydrologic Unit, and within the Wilson Creek Hydrologic Area and undefined Hydrologic Sub-Area (HSA) 103.50. The southern end of the project site is within the Lower Klamath River Hydrologic Area and the Klamath Glen HSA 105.11.

Regional groundwater flow is generally from east to west towards the Pacific Ocean (Mathany et al., 2011; Mathany and Belitz, 2015). Restrictive geologic structures such as fault zones and landslide boundaries can locally affect groundwater movement.

The region is the wettest in California, receiving an average of 70 inches of annual rainfall (DWR, 2021; Mathany and Belitz, 2015). A majority of the annual rainfall in the area occurs between October and May, with the highest monthly average in December (Caltrans, 2021a). Groundwater recharge in the project area occurs mostly from a combination of direct percolation of rainfall and infiltration from local creeks and runoff from surrounding areas. Seepage from drainage channels and tributaries of Wilson Creek also contributes to groundwater recharge (DWR, 2021).

The hydrogeology of the site area is dominated by groundwater flow along fractures in the bedrock, within the Melange and Broken Formations, and the overlying landslide deposits. The permeability of unfractured rock within these formations is low, and most groundwater occurs and is transmitted within fractures of unknown interconnection. Where water-filled fractures intersect the bluff face at the coast, groundwater discharges as a spring or seep. Groundwater is also entering the ocean below the shoreline (sub-seafloor discharge), but that volume and location(s) are unknown.

Groundwater conditions in the project area are influenced by geology, nature and geometry of landslides, and frequency and intensity of rainfall events. Groundwater flow along fractures in the project area can be interrupted and redirected, perched, or locally impounded behind subsurface barriers to flow such as clay-filled landslide-rupture zones. Hydraulic parameters have been measured at the site by packer and pumping tests; these are described in the Final Preliminary Geotechnical Data Report (Caltrans, 2022a), the Draft Hydrogeology Report (Caltrans, 2022b), and the Final Aquifer Pumping Test Technical Memorandum (Caltrans, 2022c).

An evaluation of groundwater pressure measurements, listed in Table 7 of the Final Preliminary Geotechnical Data Report (Caltrans, 2022a), suggests that groundwater may locally occur under semi-confined to confined conditions as it moves through the fracture systems within the bedrock and landslide masses. In general, however, groundwater head measurements from VWPs and the pumping tests performed at the project site suggest unconfined conditions, although additional investigation is required to more fully assess groundwater occurrence. Landslide-complex-specific groundwater conditions are described below in Section 3.3.6. Alignment-specific discussion of the groundwater is provided in Sections 3.6.4 and 3.7.4.

3.3.2 Seasonal Groundwater Variations

Most of the VWPs have been collecting groundwater data long enough to include seasonal variations, if present (Caltrans, 2022b). Seasonal variation and/or a response to rainfall is apparent in some VWP measurements; these are distinguished by the rapidity of response, i.e., rainfall response is generally immediate while seasonal response is reflected in longer-term trends. Water levels in many of the shallow open-case wells, for example, appear to exhibit a rapid response to rainfall (e.g., RC-18-003). Some deep VWPs may indicate a response to seasonal rainfall (e.g., D-20-009, 195 feet), although the response may be delayed (e.g., D-20-009, 260 feet; P-19-007, 295 feet). While most locations exhibit some response to rainfall, a few do not (Caltrans, 2022b). This suggests a complex system with varying proximity and connection to recharge areas that may include upland depressions that collect water during rainfall events, direct infiltration at the site during rainfall, and more distant recharge upgradient of the site.

3.3.3 Groundwater Levels and Hydraulic Gradients

Groundwater measurements have been collected for over three years at most locations, which includes the eight open-case wells installed in 2018. Table 3 of the Draft Hydrogeology Report presents VWP groundwater measurements from 2018 through May 2022 (Caltrans, 2022b), although the range for each VWP varies. Tables and charts in this PGR include data through June 2023, if available.

Vertical hydraulic gradients are generally downward at VWP locations. Several locations exhibit more variable vertical gradients, as follows:

- D-20-009 (three VWPs) – although a gradient between shallow and intermediate VWPs appears downward, the shallow VWP may not be submerged so is not providing meaningful data; the gradient between intermediate and deep VWPs is strongly and consistently upward.
- RC-20-014 (three VWPs) – gradient between shallow and intermediate VWPs is downward; gradient between intermediate and deep VWPs is slightly upward.
- RC-20-016 (five VWPs) – gradient is downward among the five VWPs, but groundwater levels at the two deepest VWPs are similar, and the gradient is sometimes upward.
- RC-21-001 (three VWPs) – gradient between shallow and intermediate VWPs is upward; gradient between intermediate and deep VWPs is downward; gradient between shallow and deep VWPs is upward.

The anomalous vertical hydraulic gradients at these locations may be caused by the presence of confining units within the formation and/or local changes in the flow of groundwater from recharge to discharge areas. For the portion of the Alternative F tunnel and Alternative X drainage alignments within the Melange, the groundwater is expected to be dominated by fracture flow. The data for VWP RC-20-014 indicate similar head values for the two deeper VWPs and a small upward gradient at these depths. This could indicate a confined condition locally or a local change in the vertical direction of flow. For the portion of the tunnel and drainage alignments within the Broken Formation, VWPs recorded apparent groundwater elevations above and below the alignment.

3.3.4 Observed Influence of Area Groundwater on Slope Stability

Groundwater pressure, as recorded by the network of 31 VWP installation locations (62 individual VWPs), varies by geologic unit, as well as throughout the four landslide complexes. Inclinometers installed throughout multiple phases of subsurface exploration, often concurrently with VWPs, are surveyed at regular intervals. These provide integral data regarding slope stability and landslide movement/deformation at depth.

Observations by Wills (2000) indicate that groundwater conditions affect slope stability in the area. Wills concludes that earthflows and debris flows are more likely to be affected by individual intense precipitation events, during which high pore pressures can develop and trigger movements. Further, he suggests that areas with large, deep rockslides are sensitive to long-term changes in the groundwater regime. In years with high precipitation, the regional groundwater level can rise, decreasing overall stability and triggering movements in slopes that had been stable during drier periods.

Flatter slope areas and depressions below the upper part of a rockslide or earthflow can collect and hold more water than adjacent areas of the slope, decreasing the overall stability of the slide mass beneath by slowing surface water runoff. This allows more water to infiltrate, locally raising the groundwater level or perching groundwater on low-permeability barriers.

Discussion on the observed influences of groundwater on slope stability relative to the four major landslide complexes that affect Alternatives X and F is provided in Sections 3.6.4 and 3.7.4, respectively.

3.3.5 Area Groundwater Quality

A groundwater sample was collected from open-case well P-20-012 on December 6, 2021, for chemical analyses. The groundwater sample was analyzed for compounds listed with effluent limitations pursuant to the Basin Plan (North Coast RWQCB, 2018), the Ocean Plan (SWRCB, 2019), *Monitoring Results Report: Fiscal Year 2015–2016* (Caltrans, 2016) for Areas of Special Biological Significance (ASBS) monitoring requirements, and the *Water Discharge Requirements for Discharges of Highly Treated Groundwater to Surface Waters Following Extraction and Treatment of Groundwater Polluted with Petroleum Hydrocarbons and Volatile Organic Compounds* (North Coast RWQCB, 2016). Only pH and hardness had results for comparison to the listed guidelines. The pH result (7.58) was within the permissible ranges and hardness (270 mg/L) exceeded the Basin Plan (60 mg/L) and ASBS (2.0 mg/L) limitations.

3.3.6 Groundwater Conditions at Landslide Complexes

The Alternative X and Alternative F alignments traverse the four landslide complexes: NLCG, SLCG, WC, and EF. Groundwater conditions within each landslide complex are described in the following sections. Data are presented in the Final Preliminary Geotechnical Data Report (Caltrans, 2022a).

3.3.6.1 North Last Chance Grade Complex (NLCG) Groundwater Conditions

Three VWPs were installed near the head of the NLCG landslide (Figure 3). VWPs RC-20-013 and RC-20-017 are located within the landslide mass of the NLCG and RC-19-003 is located approximately 100 feet away from the slide. Each of the transducers in VWPs RC-20-013 and RC-20-017 are located below the failure surface of the NLCG and each have measured groundwater at an apparent elevation below the landslide, suggesting that the landslide mass in the head scarp area may be unsaturated. There may also be low hydraulic connection with fractures in the underlying bedrock, which can be evaluated by collecting more site-specific data. An abrupt drop in apparent groundwater elevation between VWP RC-19-003 (Figure 4) and VWP RC-20-013 (Figure 5) suggests a possible discontinuity, or barrier, between the two locations. The NLCG failure surface (back-scarp) between VWP RC-19-003 and VWP RC-20-013 could have rapid hydraulic connection to local recharge. Although there is no groundwater monitoring within the western part of the slide mass, it is reasonable to assume rainfall onto the NLCG west of the highway has infiltrated into the fractures of the slide.

The transducers in the NLCG VWPs have recorded groundwater at different pressures (apparent elevations) since their installation (Table 4). The stratigraphy of the Broken Formation (from boring logs) suggests that locally confining conditions may occur, either due to lithology and/or occurring in separate fracture systems. The VWP data indicate a downward vertical hydraulic gradient suggestive of proximity to a discharge area.

Based on results of packer tests, hydraulic conductivity in the Broken Formation at NLCG is estimated to be 4.07×10^{-7} to 1.88×10^{-6} feet/second at depths of 170 to 180 feet and 206 to 216 feet, respectively (Table 4). Hydraulic conductivity may be locally higher or lower than indicated by packer test results, and fracture intervals are likely to have the highest conductivity.

Figure 3. Location of NLCG with VWPs, Springs/Seeps and Weather Station C

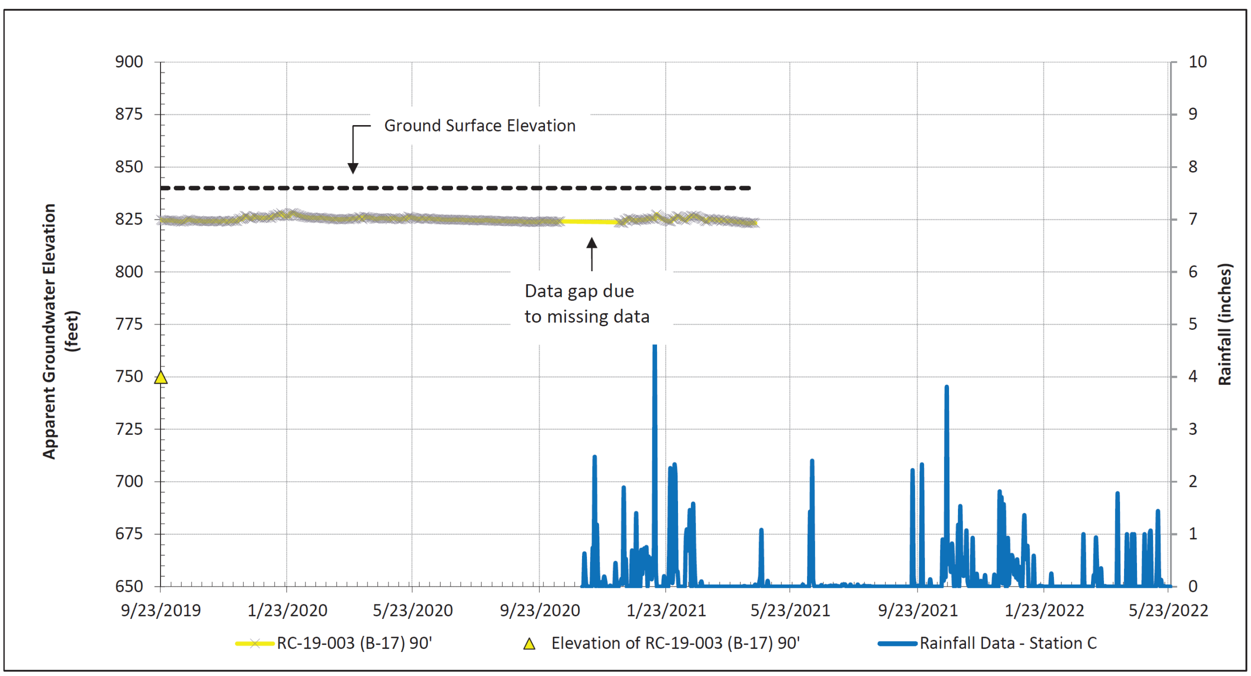


Figure 4. VWP RC-19-003 Hydrograph with Rainfall

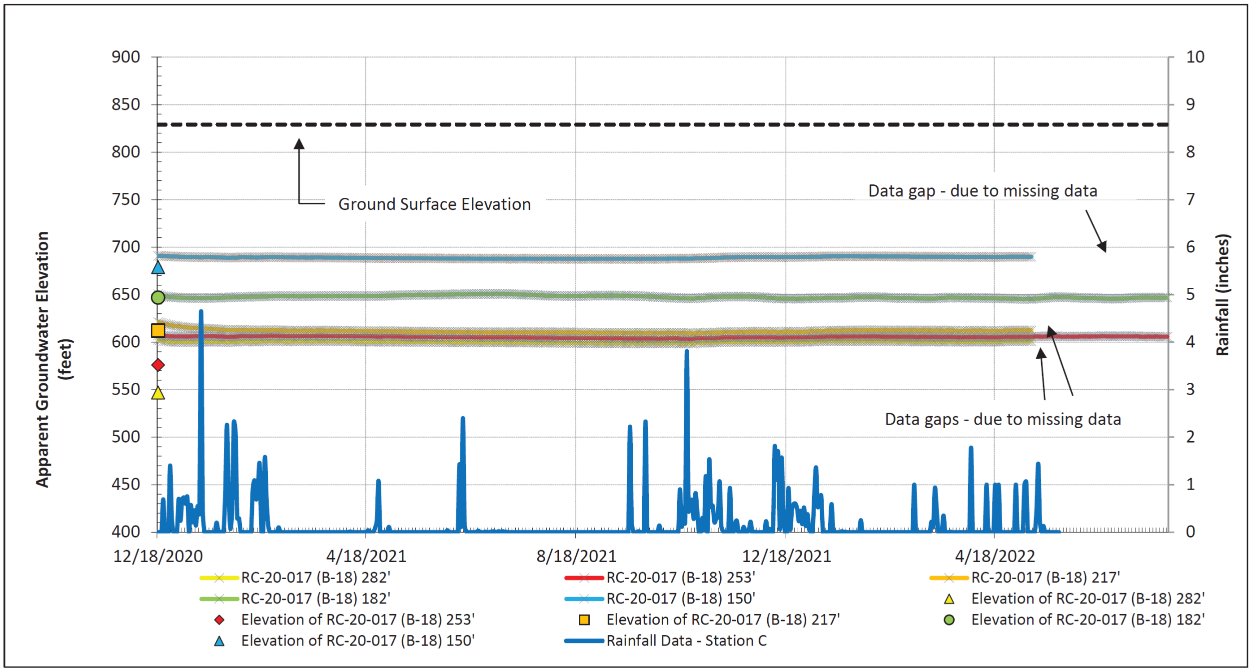


Figure 5. VWP RC-20-017 Hydrograph Showing Multiple Transducers and Rainfall

Table 4. North Last Chance Grade Landslide Complex (NLCG) Groundwater Information

Boring ID	Total Bore Depth (feet)	Surveyed Ground Surface Elevation (feet)	Packer Testing		Transducer Depth (feet)	Transducer Elevation (feet)	Apparent Groundwater Depth Minimum (feet)	Apparent Groundwater Elevation Maximum (feet)	Comments/Notes
			K value (ft/sec)	Test Interval Depth (feet)					
RC-19-003	100	840.5	--	--	90	750.5	11.6	828.9	Data through April 19, 2021.
RC-20-013	135	830.5	--	--	133	697.5	82.5	748.0	Failure surface approximately 17 feet deep (elev. 814 feet). Data through February 15, 2022.
RC-20-017	300	829.4	Failed Test	275 to 285	282	547.4	225.9	603.5	Failure surface approximately 82 feet deep (elev. 747 feet). Data through June 21, 2023.
					253	576.4	221.8	607.6	
			1.88E-06	206 to 216	217	612.4	207.5	621.9	
			4.57E-07	170 to 180	182	647.4	177.8	651.6	
					150	679.4	137.9	691.5	

3.3.6.2 South Last Chance Grade Complex (SLCG) Groundwater Conditions

Three VWPs are located within the landslide mass of the SLCG (Figure 6). VWPs P-20-002, RC-20-011, and RC-20-015 are located in the middle and upper part of the landslide near the head scarp. The transducers in VWP RC-20-015 are located below the failure surface of the SLCG; however, the transducers in VWPs RC-20-011 and P-20-002 are located above and below the landslide failure surface (Figure 6). VWP RC-20-015 is located near the head of the landslide with measured groundwater at an apparent elevation below the landslide in the two lower transducers. The shallowest transducer located near the base of the landslide measures very shallow to no groundwater present, suggesting that the landslide mass in the head scarp area may be mostly unsaturated. The remaining two VWPs, RC-20-011, and P-20-002, are located approximately 100 feet from each other near the middle of the SLCG. The VWPs measure apparent groundwater elevations from 485.3 to 736.7 feet (Table 5). Groundwater elevations indicate a downward vertical gradient at each location.

As seen on Figure 7, response to rainfall is noticeable in the two lower transducer signatures of VWP RC-20-011 but not the uppermost transducer located at a depth of 144 feet, which may not be under saturated conditions. Springs/seeps mapped along the bluff face near the beach (Figure 6) emanate from the sandstone and argillite bedding and fractures at about 1 gallon/minute (visual observation). These springs/seeps are draining groundwater from the landslide mass and/or the underlying Broken Formation.

Figure 6. Location of SLCG with VWPs, Springs/Seeps and Weather Station C

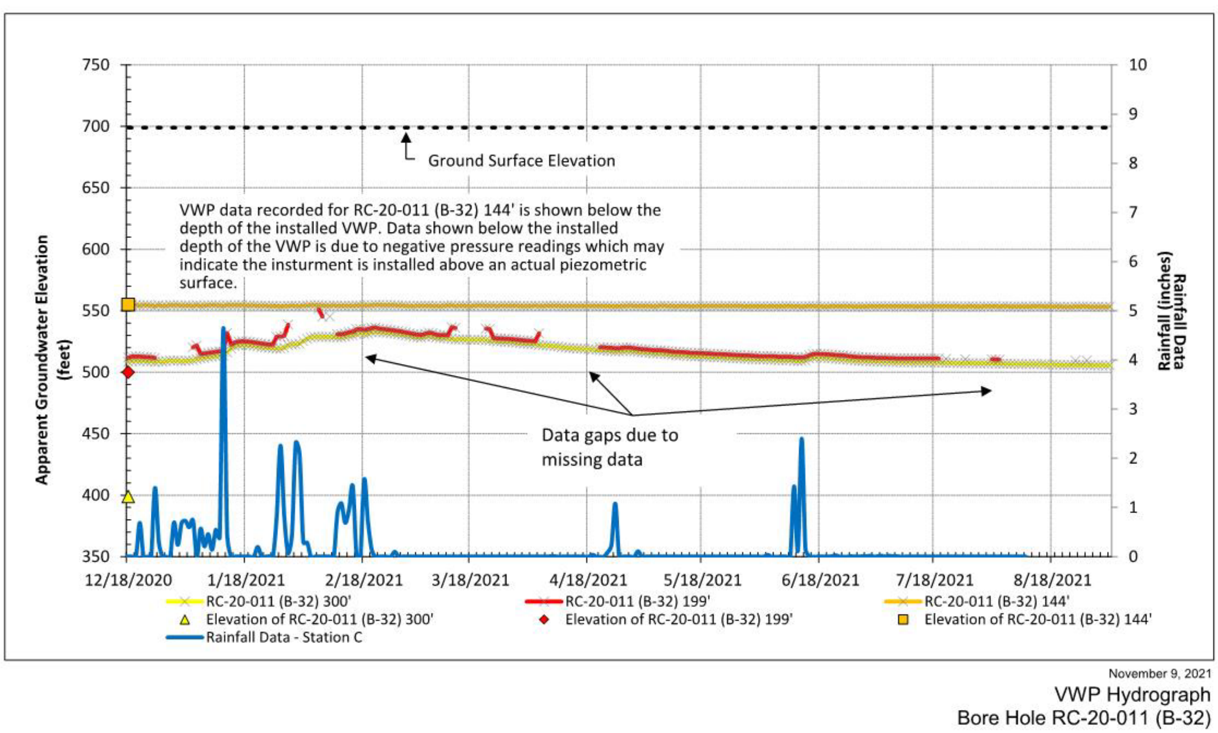


Figure 7. VWP RC-20-011 Hydrograph Showing Multiple Transducers and Rainfall

Table 5. South Last Chance Grade Landslide Complex (SLCG) Groundwater Information

Boring ID	Total Bore Depth (feet)	Surveyed Ground Surface Elevation (feet)	Packer Testing		Transducer Depth (feet)	Transducer Elevation (feet)	Apparent Groundwater Depth Minimum (feet)	Apparent Groundwater Elevation Maximum (feet)	Comments/Notes
			K value (ft/sec)	Test Interval Depth (feet)					
P-20-002	251	713.4	--	--	246	467.4	228.1	485.3	Approximately 6,500 gallons of water used at depth 180 to 200 feet.
					195	518.4	155.7	557.7	
					130	583.4	125.5	587.9	
RC-20-011	302.5	698.5	Failed Test	272 to 292	300	398.5	165.9	532.6	Failure surface approximately 261 feet deep (elev. 438 feet). Considerable water loss/used during drilling. Data through November 21, 2022.
					199	499.5	162.3	536.2	
					144	554.5	144.1	554.4	
RC-20-015	301	883.4	--	--	290	593.4	241.0	642.4	Failure surface approximately 161 feet deep (elev. 722 feet).
					255	628.4	149.4	734.0	
					159	724.4	146.7	736.7	

3.3.6.3 Wilson Creek Complex (WC) Groundwater Conditions

Five VWPs (P-19-007, RC-20-005, D-20-009, RC-20-014, and RC-20-016) were installed near the head of the WC landslide and along its northern and southern boundary (Figure 8). Multiple transduces were installed in the VWPs at various depths above and below the failure surface of the WC (Figure 8). The transducers in the VWPs have measured groundwater at numerous apparent elevations throughout the landslide mass and the underlying bedrock. The groundwater appears to be unconfined to locally confined and occurring in the fractures within the landslide and underlying Broken Formation (based on boring logs). VWP D-20-009 provides an example of an upward vertical hydraulic gradient in the WC landslide that may indicate locally confined conditions (Figure 9). The measured hydrostatic head at deepest transducer (depth 260 feet) has been generally above that measured by the two shallower transducers at depths of 95 feet and 195 feet. However, starting in August 2021, the apparent groundwater elevation measured by the deepest transducer dropped below that of the shallow transducer but recovered above this elevation by late 2021.

Infiltration of rainfall into the groundwater system is noticeable in the lower two transducer signatures of VWP D-20-009, as seen in Figure 9, while there is negligible response to rainfall in the shallowest transducer signature, which may not be under saturated conditions. Seasonal effects are also apparent in the response of the deepest VWP (260 feet) with a more subdued seasonal response in the 195-foot-deep VWP. Springs/seeps mapped along the bluff face near the beach (Figure 8) emanate from the sandstone and argillite bedding and fractures at 1 to 50 gallons per minute (visual observation).

Based on results of packer tests, hydraulic conductivity in the Broken Formation at WC is estimated to be 4.19×10^{-7} to 6.22×10^{-8} feet/second at depths of 170 to 180 feet and 206 to 216 feet, respectively (Table 6). Hydraulic conductivity may be locally higher or lower than indicated by packer test results, and fracture intervals are likely to have the highest conductivity.

Figure 8. Location of WC with VWPs, Springs/Seeps and Weather Station E

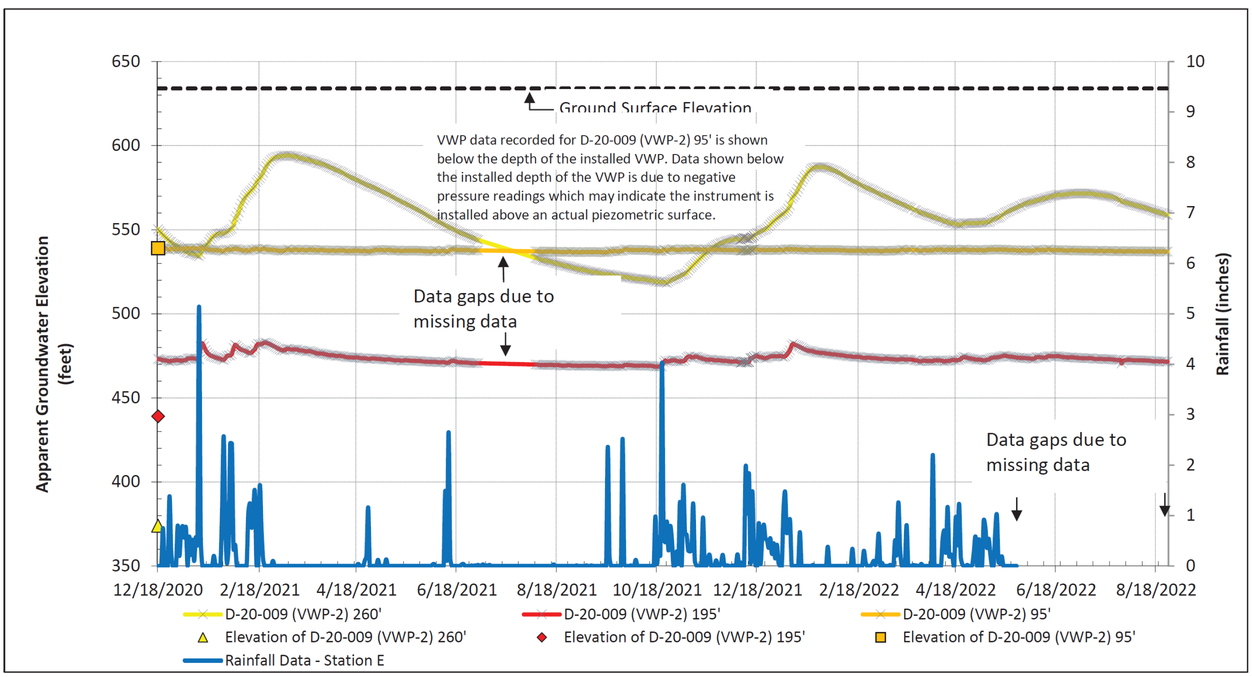


Figure 9. VWP D-20-009 Hydrograph Showing Multiple Transducers and Rainfall

Table 6. Wilson Creek Landslide Complex (WC) Groundwater Information

Boring ID	Total Bore Depth (feet)	Surveyed Ground Surface Elevation (feet)	Packer Testing		Transducer Depth (feet)	Transducer Elevation (feet)	Apparent Groundwater Depth Minimum (feet)	Apparent Groundwater Elevation Maximum (feet)	Comments/Notes
			K value (ft/sec)	Test Interval Depth (feet)					
P-19-007	Unknown	585.5	--	--	295	290.5	218.5	367.0	--
					195	390.5	146.8	438.7	
					95	490.5	82.6	502.9	
RC-20-005	250	859.1	--	--	250	609.1	216.6	642.5	Failure surface approximately 171 feet deep (elev. 688 feet).
					232	627.1	205.6	653.5	
					155	704.1	142.9	716.2	
D-20-009	265	633.8	--	--	260	373.8	39.4	594.4	Failure surface approximately 270 feet deep (elev. 364 feet).
					195	438.8	145.8	488.0	
					95	538.8	94.5	539.3	
RC-20-014	300	805.1	6.22E-08	290 to 300	290	515.1	167.0	638.1	Failure surface approximately 77 feet deep (elev. 728 feet).
			4.19E-07	220 to 230	225	580.1	167.0	638.1	
			Failed Test	163 to 173	166	639.1	147.8	657.3	
RC-20-016	300.5	674.4	--	--	287	387.4	217.5	456.9	Failure surface approximately 287 feet deep (elev. 387 feet). Mostly sandstone.
					255	419.4	217.1	457.3	
					192	482.4	191.7	482.7	
					173	501.4	172.1	502.3	
					136	538.4	134.7	539.7	

3.3.6.4 Large Earthflow Complex (EF) Groundwater Conditions

Seven VWPs (RC-18-001, RC-19-004, RC-20-006, D-20-010, RC-20-019, RC-20-020, and RC-21-001) with multiple transducers were installed near the middle of the EF landslide, with transducers located above and below the landslide failure surface (Figure 10). To clarify the greater topographic and related groundwater elevation changes across this area compared to the others, the apparent groundwater elevation ranges from a high of 561.1 feet (VWP RC-20-006) in the north to 163.7 feet (VWP RC-20-020) in the south. VWPs RC-20-019 (Figure 11) and RC-20-020 (Figure 12), show typical signatures from transducers located below and above the EF landslide mass. The apparent groundwater elevations illustrated on Figure 11 and Figure 12 indicate that groundwater within the landslide mass and underlying Melange may have a hydraulic connection. Boring logs suggest groundwater occurs within the fractures of the clayey Melange.

Other groundwater measurements in the EF VWPs also indicate that the groundwater is locally confined. For example, both transducers at VWP D-20-010 have measured apparent groundwater elevations 8.3 to 10.1 feet above the ground surface. Since no manifestation of surface discharge has been identified, the groundwater may be confined, and groundwater tapped at the depth of these transducers might manifest with artesian conditions. Apparent groundwater elevations are similar at the two transducers, one at the base of the landslide and one below the failure surface (Figure 13). These similar apparent groundwater elevations also indicate the fracture systems influencing the two transducers may be hydraulically connected.

A response to rainfall at VWPs RC-20-019 and RC-20-020 is noticeable in the two upper transducer signatures but not the two lower transducers. This may be due to a lack of hydraulic connection between the upper and lower fracture systems. Also, rainfall events are not noticeable on either of the transducer signatures for VWP D-20-010 (Figure 13), suggesting that more hydrogeologically isolated conditions at depth may be buffering significant effects of recharge events. A modest seasonal effect is apparent for the shallow VWP of RC-20-019.

Although there is no groundwater monitoring in the eastern and western part of the slide mass, it is reasonable to assume rainfall onto the EF has infiltrated into the fractures of the landslide. West of U.S. 101, mapped springs/seeps along the slope face near the beach (Figure 10) drain groundwater from the landslide mass and/or the underlying Melange at an estimated rate of 0.5 to 5 gallons per minute (visual observation).

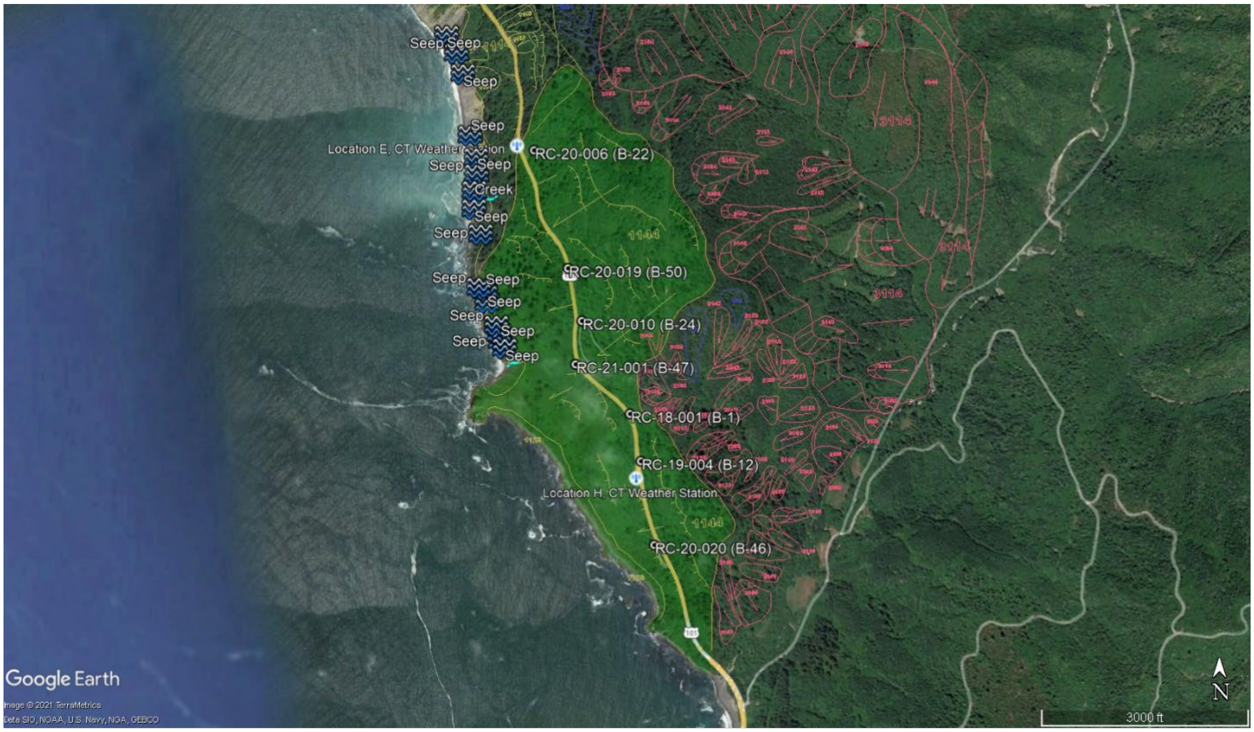


Figure 10. Location of EF with VWPs, Springs/Seeps and Weather Stations E and H

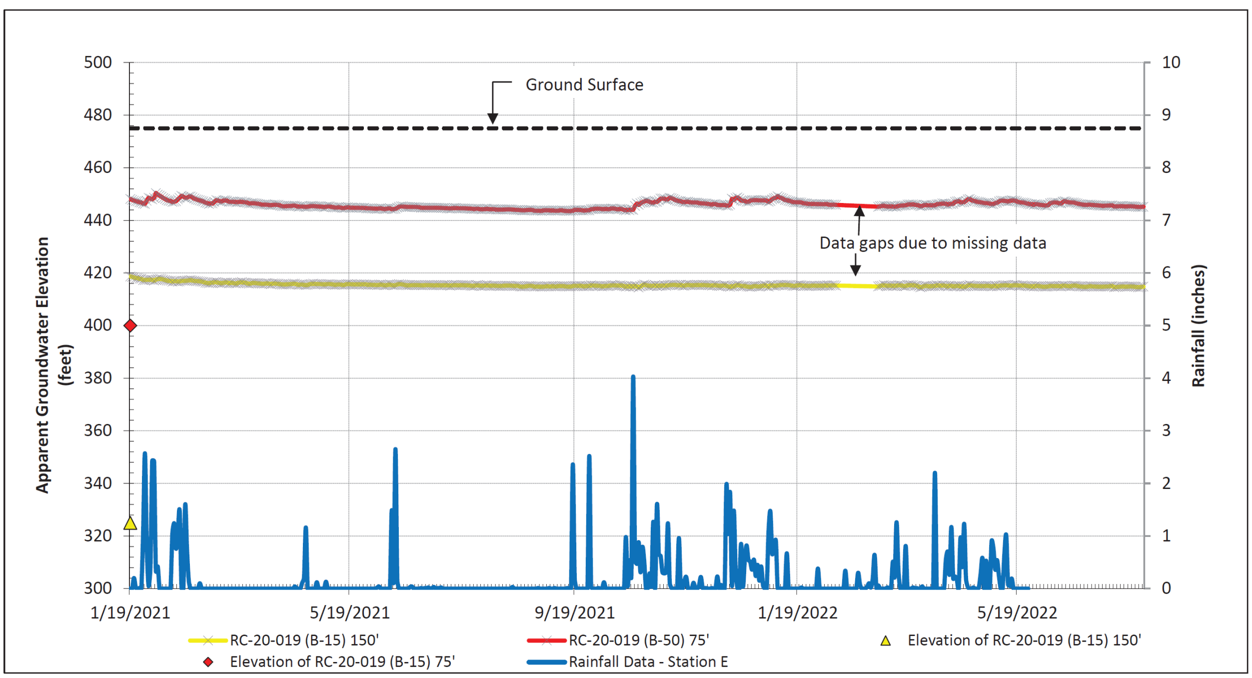


Figure 11. VWP RC-20-019 Hydrograph Showing Multiple Transducers and Rainfall

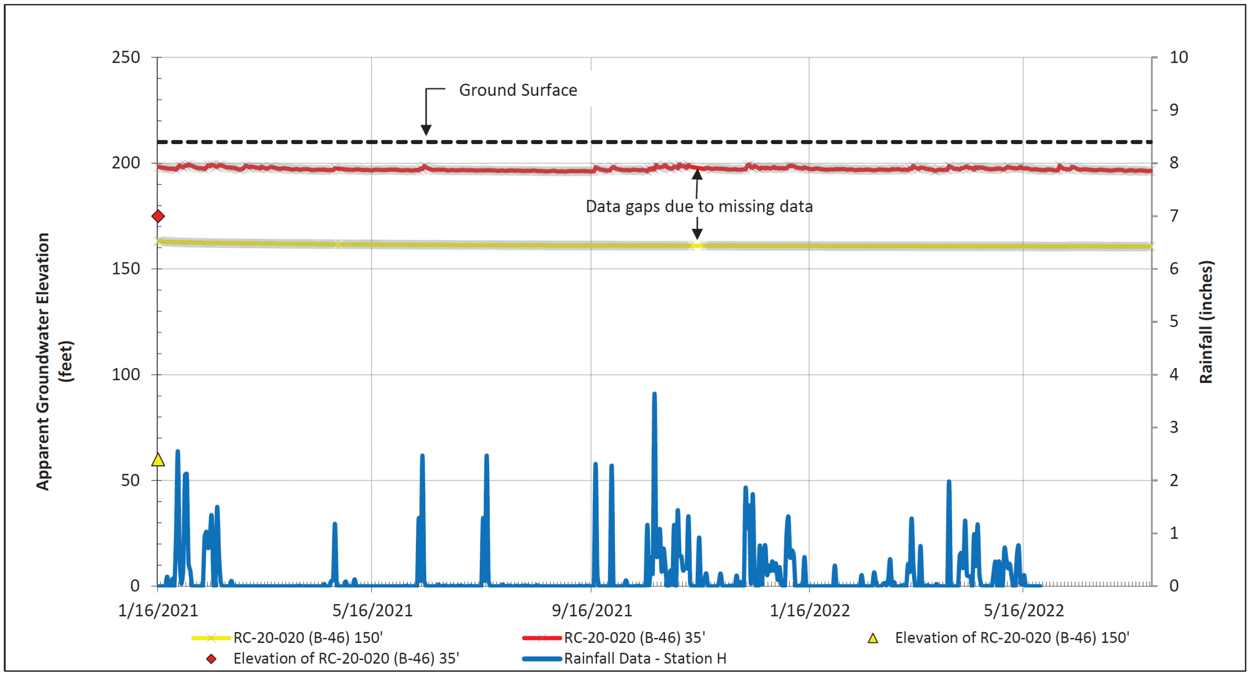


Figure 12. VWP RC-20-020 Hydrograph Showing Multiple Transducers and Rainfall

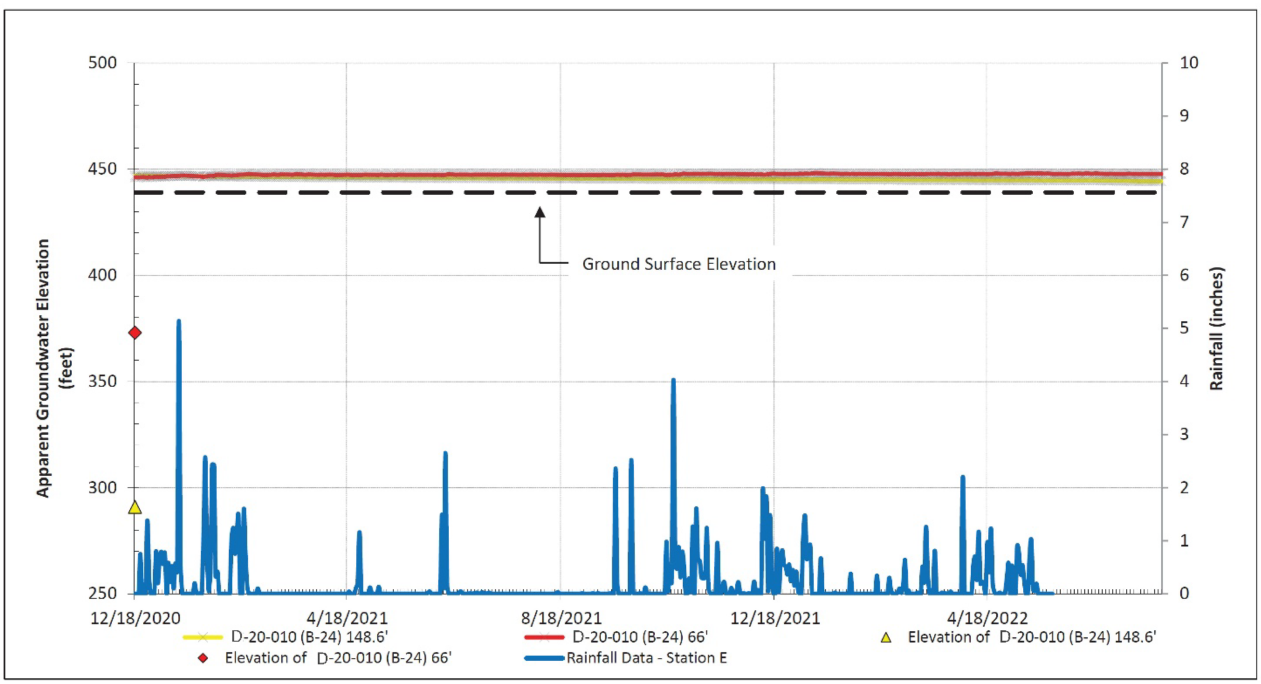


Figure 13. VWP D-20-010 Hydrograph Showing Multiple Transducers and Rainfall

Table 7. Large Earthflow Complex (EF) Groundwater Information

Boring ID	Total Bore Depth (feet)	Surveyed Ground Surface Elevation (feet)	Transducer Depth (feet)	Transducer Elevation (feet)	Apparent Groundwater Depth Minimum (feet)	Apparent Groundwater Elevation Maximum (feet)	Comments/Notes
RC-18-001	85	345.1	69.8	275.3	5.5	339.6	Failure surface approximately 67 feet deep (elev. 279 feet). Data through January 4, 2022.
RC-19-004	100	289.4	48.5	240.9	4.4	285.0	Failure surface approximately 49 feet deep (elev. 240 feet). Data through February 15, 2021.
RC-20-006	251	619.3	199.5	419.8	73.5	545.8	Failure surface approximately 80 feet deep (elev. 539 feet).
			129	490.3	65.3	554.0	
			60	559.3	58.2	561.1	
D-20-010	150	438.9	148.6	290.4	-8.3	447.3	Failure surface approximately 67 feet deep (elev. 372 feet). Groundwater encountered at 15 feet during drilling and exhibited artesian conditions.
			66	372.9	-10.1	449.0	
RC-20-019	151	474.7	150	324.7	56.0	418.7	Failure surface approximately 143 feet deep (elev. 332 feet).
			75	399.7	24.3	450.4	
RC-20-020	150	210.4	150	60.4	46.7	163.7	Failure surface approximately 62 feet deep (elev. 148 feet).
			35	175.4	9.6	200.8	
RC-21-001	150	408.4	149	259.4	23.7	384.7	Failure surface approximately 98 feet deep (elev. 310 feet). Data through February 11, 2023.
			49	359.4	8.7	399.7	
			30	378.4	26.7	381.7	

3.4 Area Soil/Rock Units – Geotechnical Properties

Subsurface materials in the project area comprise landslide deposits, failure zone materials, and bedrock formations. Table 8 presents descriptions and properties of these materials. Strength parameters shown in Table 8 for use in slope stability analyses were based on data collected during the Phase 2B investigation and then verified and/or calibrated through back analysis during slope stability analysis, as discussed in Section 4. Laboratory test data are presented in the Final Preliminary Geotechnical Data Report (Caltrans, 2022a).

Geotechnical properties of site subsurface materials specific for tunneling are discussed in Section 3.5.

Area landslide deposits comprise local colluvial deposits (Qc), landslide deposits derived from Franciscan Complex Melange (Qlsd-m), and landslide deposits derived from Franciscan Complex Broken Formation (Qlsd-bf). Materials in the landslide complexes and can be divided into two general types:

- Soil and soil-like material includes colluvium, alluvium, and the soil matrix and failure zones within landslide deposits derived from Franciscan Complex Melange and Broken Formation. Gravel-size rock fragments are here considered “soil-like.”
- Intact rock material includes cobbles, boulders, and rock blocks of various sizes within the landslide deposits, where “block” is here defined as a volume of intact rock which is significantly stronger than the material surrounding it.

Two different types of failure zone materials are present in the area. The earthflow basal failure zone (Fs-1) is either a discrete clay-rich zone or a zone of brecciated shale fragments at the base of the earthflow landslide deposits. Rock/debris landslide failure zones (Fs-2) are narrow, sheared zones of weakness along which sliding occurs within and at the base of rock/debris landslide deposits.

The two bedrock formations in the project area are the Franciscan Complex Melange (KJFm) and the Franciscan Complex Broken Formation (KJFbf).

Melange in the project area consists of dark gray, pervasively sheared, soil-like argillite with scattered blocks of intact sandstone. Sandstone blocks can be tens of feet across in maximum dimension.

Broken Formation rocks in the project area consist of blocks of gray, hard, massive to very thickly bedded sandstone with interbedded argillite separated by weak, sheared zones mainly of thickly bedded, gray sandstone with lesser siltstone and shale interbeds.

Table 8. Geologic Units and Strength Parameters

Symbol	Geologic Unit	Description	Lithology	Total Unit Weight	Internal Friction Angle (ϕ)	Cohesion (c)
				pcf	degrees	psf
Qal	Alluvium	Sand and sandy gravel with some fine-grained soil	*combined with colluvium (Qc)*	-	-	-
Qc	Colluvium	Loose, heterogeneous mass of soil material and/or rock fragments transported and deposited downslope by sheet flow or slow, continuous creep	mixed (gravel to clay)	120 to 125	25 to 26	50
Qlsd-m	Earthflow Landslide Deposits, Derived from Melange	Landslide deposits consisting of a mixture of fine-grained soils, deeply weathered rock, and scattered sandstone clasts which have been transported as a sliding mass with many internal slip surfaces. Includes sandstone blocks tens of feet across in intensely sheared argillite matrix	argillite with sandstone clasts	130 to 140	26	250 (1)
Qlsd-bf	Rock/Debris Landslide Deposits, Derived from Broken Formation	Landslide deposits consisting of blocks of sandstone with minor argillite rock and/or debris which have been transported by sliding or falling	sandstone/argillite	140	36 to 45	1000 to 3000
Fs-1	Earthflow Basal Failure Zone(s)	Discrete clay-rich sliding zone at the base of Earthflow Landslide Deposits	sheared argillite	140 to 145	18 to 30(1)(2)	0
Fs-2	Rock/Debris Landslide Failure Zone(s)	Narrow, sheared zones of weakness along which sliding occurs within and at the base of Rock/Debris Landslide Deposits	sheared sandstone/argillite	140 to 145(2)	32 to 34(1)	0
KJFm	Franciscan Complex, Melange	Dark gray, pervasively sheared, soil-like argillite with scattered blocks of intact sandstone. Sandstone blocks can be tens of feet across	argillite	140 to 145	28	500
KJFbf	Franciscan Complex, Broken Formation	Blocks of gray, hard, massive to very thickly bedded sandstone with interbedded argillite separated by weak, sheared zones	interlayered sandstone/argillite	140 to 155	40 to 45	1000 to 3000

Notes:

(1) Strength parameters were estimated through back analysis of existing slope configurations and groundwater conditions.

(2) The lower value is based on a single, continuous basal plane. The upper value is based on multiple basal slide planes.

3.5 Area Tunneling Conditions

3.5.1 Ground Classification for Tunneling

A Preliminary Tunnel Ground Classification System was developed for LCG tunneling, including the drainage gallery tunnels proposed for Alternative X and the highway tunnel proposed for Alternative F. The ground classification criteria consider the site geologic setting, the nature and variability of ground conditions affecting tunnel design, and the probable construction methods to be used. The Preliminary Tunnel Ground Classes and their distinguishing characteristics are summarized in Table 9. Criteria for ground classification are based on data presented in the Final Preliminary Geotechnical Data Report (Caltrans, 2022a) and will be refined as additional data are collected.

Table 9. Preliminary Tunnel Ground Classification System

	Ground Class Group	Geologic Symbol	Ground Class	Distinguishing Characteristics (1)(2)(3)(4)(5)(6)
Overburden	Fill/Colluvium/ Alluvium	F	Fill	• Variable clayey sand, silty sand with gravel, coarse to fine sand, or sandy clay
		Qal	Alluvium	• Loose sand, silt, and gravel; locally includes clay from undercut stream banks in Earthflow and Melange
		Qc	Colluvium	• Loose, heterogeneous mass of soil material and/or rock fragments transported and deposited downslope by sheet flow or slow, continuous creep • Sandy silt, silty sand, clayey sand, clayey gravel with roots and subangular coarse to fine sandstone fragments • Locally mixed with residual soil • Recoverable with soil sampling equipment; drive samples generally possible
Landslide Deposits	Landslide Deposits	Qlsd-m	V	• Earthflow landslide deposits, derived from Melange and transported as a sliding mass with many internal sheared surfaces • Mixture of fine-grained soils, moderately to intensely weathered argillite and sandstone(2), and poorly graded gravel with sand • Scattered sandstone clasts (blocks(1)) less than 3 inches in maximum dimension(5) • <25% sandstone by volume • Recoverable by soil sampling methods; drive samples generally possible • No visible fabric or relict rock structure • Includes discrete clay-rich sliding zone at base (Rs-1) • Can include blocks of sandstone or argillite up to 40 feet in maximum dimension.(6)
		Qlsd-bf	IV	• Rock/debris landslide deposits, derived from Broken Formation • Mixture of fractured rock, soil, and vegetation transported by sliding or falling • Non-interlocking blocks(1) of intact yellow-brown and gray sandstone and argillite up to 12 inches in maximum dimension (5) • >50% sandstone by volume • Moderately weathered to decomposed(2) • Includes intermixed zones 6 to 12 inches thick of sandy clay and fine sand from decomposed rock • Sampled by coring but includes zones which can be sampled with soil sampling methods • Generally, core recovery >50%, RQD = 0 to 25% • Includes sheared zones of weakness along which sliding occurs within and at base (Rs-2)
Bedrock	Franciscan Complex, Melange	KJFm	III	• Dark gray, fine-grained argillite with scattered clasts (blocks(1)) of intact sandstone up to 12 inches in maximum dimension(5) • Sandstone clasts (blocks(1)) randomly arranged within matrix of sheared argillite • <25% sandstone by volume • >75% matrix, <25% block • Moderately weathered to decomposed(2) • Sheared zones throughout • Completely crushed to very blocky and seamy(4) • Heavily broken rock mass with no interlocking blocks • Generally, core recovery >75%, RQD = 0 • Cannot be sampled by soil-sampling methods • Can include embedded blocks of sandstone or argillite up to 40 feet in maximum dimension. (6)
	Franciscan Complex, Broken Formation	KJFbf	II	• Blocks(1) of dark gray fine-grained sandstone with thin interbeds of argillite surrounded by matrix of sheared and partly decomposed argillite • >75% sandstone by volume • >75% block, <25% matrix • Slightly weathered to intensely weathered; locally decomposed(2) • Intact sandstone medium strong to very strong • Intact argillite very weak to medium strong • Maximum block dimension = 2 to 6 feet(5) • Fracture spacing ¾ inch to 12 inches(3) • Very blocky and seamy(4) • Blocks are poorly interlocked within partially disturbed rock mass • Generally, core recovery >75%, RQD = 0 to 50% • Cannot be sampled by soil sampling methods
			I	• Blocks(1) of dark gray, fine-grained sandstone with thin argillite beds surrounded by sheared but chemically intact argillite • >75% sandstone by volume • >75% block, <25% matrix • Slightly weathered to moderately weathered(2) • Intact sandstone medium strong to very strong • Intact argillite weak to medium strong • Maximum block dimension = 6 to 12 feet(5) • Fracture spacing ¾ inch to 3 feet (3) • Moderately blocky and seamy(4) • Blocks are partly interlocked within partially disturbed rock mass • Multifaceted angular blocks formed by four or more joint sets • Generally, core recovery >75%; RQD ≥ 50% • Cannot be sampled by soil sampling methods.

Notes:

(1) Block is here defined as a rock mass significantly stronger than the surrounding material. Block and matrix are defined based on relative strength and block size, not necessarily rock type.

(2) Weathering grades are from Caltrans, 2010, *Soil and Rock Logging, Classification, and Presentation Manual, 2010 Edition,* Division of Engineering Services, Geotechnical Services, 90 p.

(3) Fracture spacing applies to fractures with minimum persistence of 3 feet.

(4) Terzaghi rock mass description from Proctor, R.V. and T L. White, 1968, *Rock Tunneling with Steel Supports, Revised*, Commercial Shearing and Stamping Company, Youngstown, Ohio.

(5) Geologic mapping indicates that block size can be larger than indicated from core samples.

(6) Large blocks of intact rock in Earthflow deposits and Melange are not distinguishing characteristics of these units but have been observed and reported in the project area.

The Preliminary Tunnel Ground Classification System would be carried forward through project design and construction phases, with revisions as required by additional data and design changes. Key requirements considered for the classification system were:

- Applicable to anticipated construction methods, including TBM, SEM, and methods for constructing retained excavations and support structures,
- Quantitative, objective, and based on subsurface data collected and presented in current and future project geotechnical data reports,
- Standardized terminology,
- Unambiguously communicable in terms of baseline values,
- Baseline classifications can be verified during construction, and
- Classifications are objective and repeatable during both design and construction (different workers assign the same classifications).

As shown in Table 9, eight defined ground classes were grouped into four Ground Class Groups, as follows:

- Fill/Colluvium/Alluvium
- Landslide Deposits
- Franciscan Complex, Melange
- Franciscan Complex, Broken Formation

Ground Class Groups for bedrock consist of Franciscan Complex Melange and Franciscan Complex Broken Formation. The Broken Formation is further divided into two Ground Classes based on inferred block size, weathering, and strength.

Because of the chaotic distribution of materials within both the Melange and the Broken Formation, it was assumed that the mix of materials encountered in boreholes will be the same as that encountered in tunnel excavation, at least within a single reach. However, in recovered core, fracture spacing appears closer and inferred block size smaller than observed in geologic mapping. Dimensions in Table 9 are interpretive and reflect adjustments for effects of drilling, stress relief, and mapping bias in favor of better-quality rock.

For the Landslide Deposits Ground Class Group, distinguishing characteristics of the two Ground Classes are linked to their derivation from either Franciscan Complex Melange or from Franciscan Complex Broken Formation.

For Overburden, distinguishing characteristics of the three Ground Classes are related to their identification as Fill, Colluvium, or localized Alluvium.

As additional data are collected, ground classes may be grouped or subdivided differently.

3.5.2 Geotechnical Properties of Area Materials in Underground Excavations

The following sections describe the geotechnical properties of the site materials that will be encountered in underground construction proposed for Alternative X and for Alternative F. Descriptions are based on data presented in the Final Preliminary Geotechnical Data Report (Caltrans, 2022a) and supported by published reports cited in Section 3.1. Additional geotechnical properties of site materials are presented in Section 3.4. Hydraulic properties of site materials are discussed in Section 3.3.5.

Site subsurface materials along the Alternatives X and F alignments can be divided into three general types for underground construction: soil, intact rock, and Intermediate Geo-Materials (IGM).

3.5.2.1 Soil

Soil material includes Overburden deposits (Fill, Colluvium, and Alluvium) and portions of the Landslide Deposits. These materials can be expected to exhibit typical soil behavior, and properties can generally be estimated from field data and laboratory test results.

3.5.2.2 Intermediate Geo-Materials (IGM)

IGM includes material that is neither soil nor rock and exhibits ground behavior which may be different from both. IGM typically has relict rock structure, including foliation, bedding, and joints.

Soil-like IGM to be encountered in underground excavations would consist of matrix material, highly weathered to completely decomposed argillite, and fracture aperture fillings in the Earthflow, the Melange, and the Broken Formation. All the rock material is essentially decomposed to soil and can generally be sampled by drive sampling. Samples typically disintegrate when agitated in water, demonstrated its potential for slaking.

Rock-like IGM to be encountered in underground excavations would consist of highly sheared matrix material and failure zones. It is generally sampled by coring, but typically has less than about 50 percent moderately weathered rock, irregularly distributed in a soil matrix.

Both types of IGM can exhibit raveling or flowing behavior during underground excavation, especially if saturated. They are also prone to sudden collapse, as clay-coated relict fractures or joint sets allow sliding or toppling failures. IGM is difficult to sample and to characterize, especially in geologic settings as disturbed as LCG. Additional sampling and testing are required to better characterize IGM in the project area.

3.5.2.3 Rock

Intact rock material includes blocks of various sizes, where “block” is here defined as a volume of intact rock which is significantly stronger than the material surrounding it. Intact rock includes the large interlocking blocks of the Broken Formation, which are separated by filled fractures, sheared zones, and other rock mass discontinuities of varying thickness and persistence. It also includes blocks of various sizes in the Melange and the cobble-size and larger clasts and rock fragments within the Landslide Deposits.

Because most excavation for the Alternative X drainage gallery tunnels and the Alternative F highway tunnel would be in rock, intact rock properties and rock mass properties are discussed in more detail in the following sections.

3.5.3 Intact Rock Properties

Geotechnical properties of intact rock given here are based on results of laboratory tests presented in the Final Preliminary Geotechnical Data Report (Caltrans, 2022a). Intact rock properties discussed here apply only to discrete samples of intact rock. They differ from the rock mass properties discussed in Section 3.5.4, which consider the bulk properties of the rock mass as a whole.

Most material to be affected by tunneling for construction for Alternative X or Alternative F will be argillite or sandstone of the Melange and the Broken Formation. Table 10 summarizes results of laboratory tests performed on argillite samples as presented in the Final Preliminary Geotechnical Data Report (Caltrans, 2022a). Although argillite samples were typically recovered with rock coring equipment, there was little suitable sample from which specimens for rock strength tests could be prepared. The argillite is anticipated to behave more as a soil-like material than as intact rock, especially where sheared or weathered. Additional sampling and testing may confirm if this is in fact the in-situ condition of argillite or if sampling, stress relief, or other factors affected its condition.

As shown in Table 10, the slake durability index for tested argillite averaged about 68 percent. If confirmed by additional testing, this relative low durability suggests that argillite exposed during excavation will need to be sealed or covered quickly to prevent deterioration.

Sandstone constitutes the majority of rock to be encountered in excavations for both Alternative X and Alternative F, and its intact rock properties will affect selection of excavation equipment and excavation progress.

Table 11 summarizes results of laboratory tests performed on sandstone samples. Figure 14 is a percentile plot of results of unconfined compressive strength tests on sandstones.

Table 10. Summary of Intact Rock Properties for Argillite

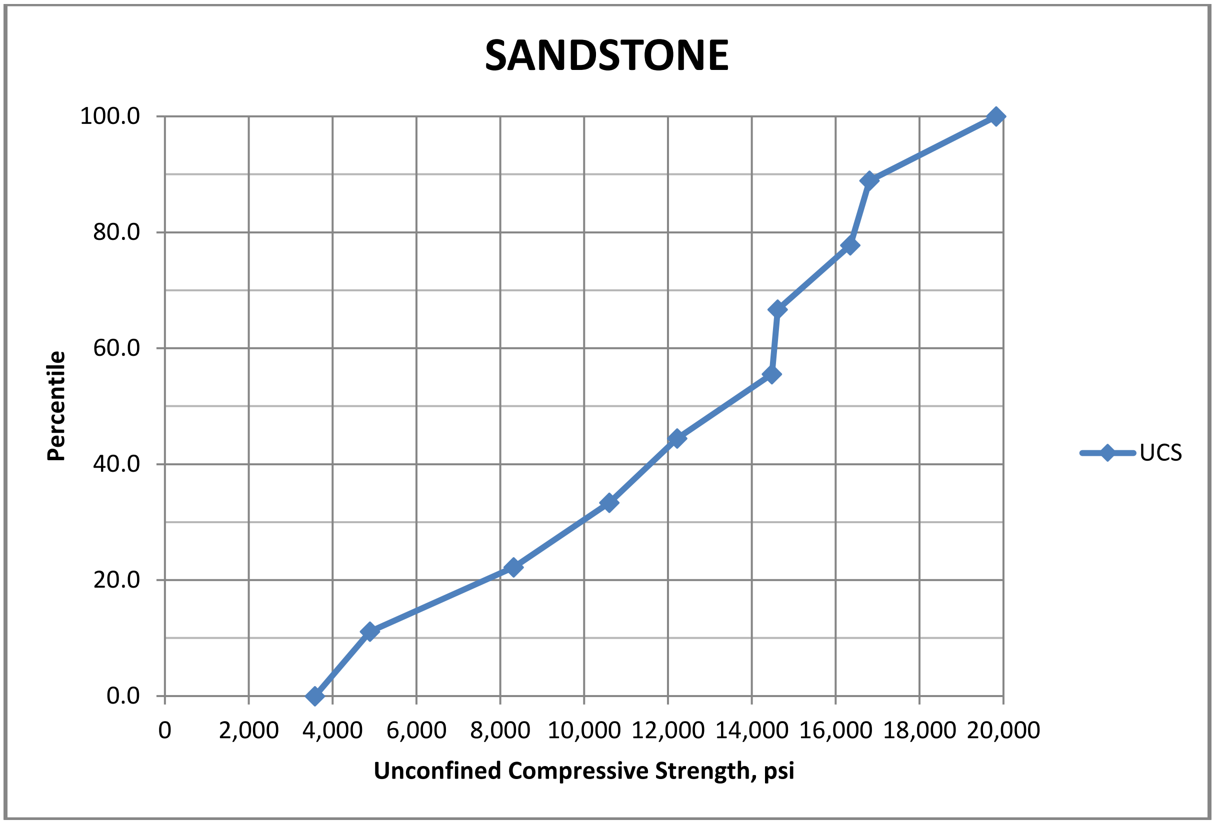
Property		Range		Average Value	Median Value	N, number of tests
		Minimum	Maximum			
Index Properties	Dry Unit Weight, pcf	134	144	137	137	7
	Water Content, %	1.2	8.6	5.8	7.2	9
Strength & Mechanical Properties	Undrained Shear Strength, psf, from UU Tests by ASTM D2850	1,870	13,300	6,481	6,113	6
	Direct Shear by ASTM D3080					
	• Cohesion, psf	1,152	1,152	1,152	1,152	1
	• Friction angle, degrees	33.2	33.2	33.2	33.2	1
Abrasiveness & Hardness	CERCHAR Abrasiveness Index	1.62	1.62	1.62	1.62	1
	Bulk Mohs Hardness	3.6	3.6	3.6	3.6	1
Slaking Properties	Slake Durability Index, %	41.0	94.8	67.9	67.9	2

Data source: July 2022 Final Preliminary Geotechnical Data Report (Caltrans, 2022a).

Table 11. Summary of Intact Rock Properties for Sandstone

Property		Range		Average Value	Median Value	N, number of tests
		Minimum	Maximum			
Index Properties	Bulk Density, pcf	167	173	168	168	10
Strength & Mechanical Properties	Unconfined Compressive Strength, psi					
	• from tests by ASTM D7012-C and D7012-D	3,580	19,830	12,170	13,350	10
	• estimated from axial PLI tests	2,610	12,420	7,105	6,695	4
	Splitting Tensile Strength, psi	560	2,970	1,484	1,280	7
Abrasiveness & Hardness	CERCHAR Abrasiveness Index	1.54	2.77	2.20	2.30	5
	Bulk Mohs Hardness	4.4	6.0	5.7	5.9	6
Slaking Properties	Slake Durability Index, %	72.7	88.2	82.8	87.6	3

Data source: July 2022 Final Preliminary Geotechnical Data Report (Caltrans, 2022a).



Notes:
1. Plot shows results of tests performed on sandstone at as-received moisture content.
2. Data source: July 2022 Final Preliminary Geotechnical Data Report (Caltrans, 2022a).

Figure 14. Percentile Distribution of Unconfined Compressive Strength Test Results for Sandstone

As shown in Table 11 and Figure 14, unconfined compressive strength of sandstone is variable, ranging from about 3,600 psi to nearly 20,000 psi, and averaging about 12,000 psi. Splitting tensile strength is relatively high (average about 1,500 psi). The relatively high quartz content indicated by bulk Mohs hardness suggests that the sandstone could be abrasive to excavating equipment. Additional testing would confirm these properties.

3.5.4 Rock Mass Properties

3.5.4.1 Rock Mass Mechanical Properties

The interaction of blocks of intact rock and the discontinuities, or matrix materials, which separate them will strongly influence the behavior of the rock mass in response to excavation for retained cuts, TBM and SEM construction, and cross passages. Conventional rock mass quality indices such as Q (Barton et al., 2002), Geological Strength Index (GSI) (Marinos et al., 2005), or Rock Mass Rating (RMR) (Bieniawski, 1992) could not reliably be applied because there are an insufficient number of borings along the Alternative F alignment, especially considering the variable ground conditions.

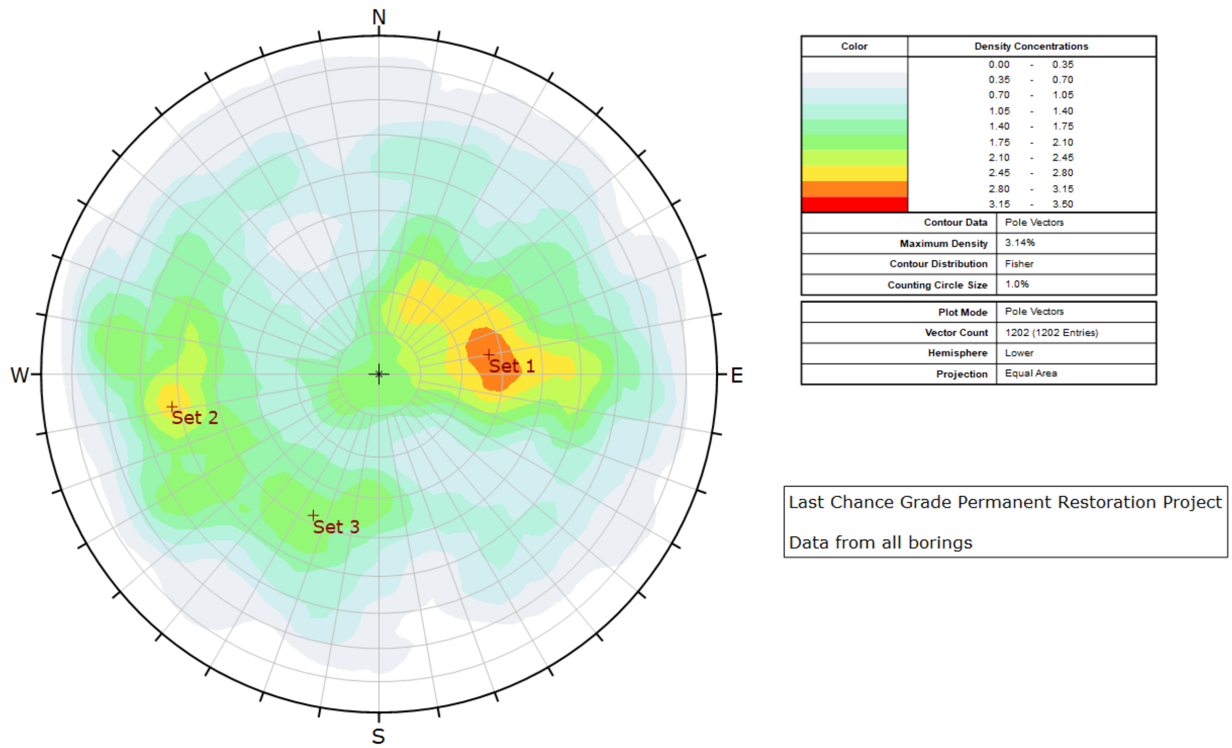
Rock Quality Designation (RQD) (Deere and Deere 1988) was used as a general indicator of rock mass quality for underground construction. As shown in the RQD ranges given for the various Tunnel Ground Classes in Table 9, RQDs are relatively low. Even in the best quality rock, Ground Class I Broken Formation, RQD was seldom over 75 percent.

Although here considered bedrock, the Franciscan Complex Melange is generally a chaotic mix of sandstone and argillite blocks of various sizes in a fractured and sheared matrix. Additional explorations might indicate grossly traceable units or structural trends that could be used to predict rock mass conditions to be encountered in tunneling.

3.5.4.2 Rock Mass Discontinuities

Rock mass discontinuities such as contacts and faults cannot yet be reliably identified or described from the current data set. Bedding is visible in the sandstone, but there are too few measurements at this time to make conclusions regarding potential influence of bedding on rock mass behavior. Especially in the Melange, fractures are likely to be randomly oriented.

The limited ATV and OTV data from downhole logging of seven boreholes suggest that some identifiable joint sets are present. Figure 15 presents a DIPS-generated (Rocscience, 2021a) lower-hemisphere equal area stereonet plot of poles to 1202 discontinuities logged in Borings RC-20-005, RC-20-011, RC-20-014, RC-20-016, RC-20-017, RC-20-019, and RC-21-001. Three joint sets were identified in this global plot and are listed in order of decreasing pole concentrations in Figure 15.



Joint Set	Dip Angle, degrees	Dip Direction, Azimuth
Set 1	27	260
Set 2	52	81
Set 3	38	25

Notes:

1. ATV/OTV data are from Borings RC-20-005, RC-20-011, RC-20-014, RC-20-016, RC-20-017, RC-20-019, and RC-21-001, as presented in July 2022 Final Preliminary Geotechnical Data Report (Caltrans, 2022a).
2. Poles per 1 percent stereonet area were counted and contoured using DIPS v.8.011 (Rocscience, 2021a).
3. Orientations of joint sets were defined as the mean plane orientation within a window of pole concentrations identified by cluster analysis using DIPS. In most cases, this orientation is similar to the polar point maximum.

Figure 15. Orientation of Fractures and Joint Sets.

Although degree of scatter varies widely by depth and by borehole, at least a small number of fractures of Joint Set 1 appear to be present in most of the logged borings. Joint Set 2 is orthogonal to Joint Set 1. Joint Set 3 is not well defined in the global stereonet plot but may be more concentrated locally.

Fractures of Joint Set 1 are locally subparallel to the slope face, possibly either as a cause or effect of past slope movements. Its strike is subparallel to the trend of the Alternative X drainage gallery tunnels and the Alternative F highway tunnel. For the SEM construction planned for Alternative F, the orientation of Joint Set 1 could pose a problem with fallout on the east wall of the excavation. Open cuts in rock at the South and North Portal could have similar east wall instability due to sliding along Joint Set 1 if not retained.

Few steeply dipping fractures were recorded, possibly because of sampling bias from the vertical borings, or because of disturbed nature of the rock mass. The orientation, persistence, and character of rock mass discontinuities and their potential effects on design and construction would be better understood with additional analysis of existing discontinuity data as well as collection of additional discontinuity data. Such additional analysis and data would enhance understanding of site geology as well.

3.5.5 Subsurface Gases

Based on currently available data, potentially hazardous subsurface gases are not anticipated to be encountered in underground excavations for Alternative X or Alternative F. Such gases might include natural occurring methane and hydrogen sulfide or gases from human-generated waste or leaking utility lines.

Review of publications by the USGS, CGS, and publicly available well data from the California Geologic Energy Management Division (CalGEM) did not yield relevant information about the potential presence of hazardous gases in the Franciscan Complex in the project area.

Gas has been reported within the Franciscan Complex within the Diablo Range of Central California, roughly 350 miles to the south of the LCG area. In the pre-construction geology report for the Pacheco Tunnel (USBR, 1976) located near Pacheco Pass along Highway 152, hydrogen sulfide and methane were detected in water and gas samples collected from test borings prior to construction.

Should hazardous subsurface gases be suspected, tunnel construction would be classified as gassy or potentially gassy. Underground equipment would be explosion resistant, and appropriate OSHA safety protocols would be incorporated in specifications.

Groundwater sampling and analysis for dissolved gases and other field tests would be required to better evaluate the potential for hazardous subsurface gases at the site.

3.6 Site Geotechnical Conditions – Alternative X

3.6.1 Site Surface Conditions – Alternative X

3.6.1.1 Existing and Proposed Above-Ground Structures, Facilities, and Utilities

U.S. 101 is the primary above-ground facility within the Alternative X area. Within the project limits, U.S. 101 consists of paved highway with two to four lanes of traffic. The highway is paved with an asphaltic concrete surface and includes a scenic overlook, pullouts (gravel, dirt, and paved), and highway signs and signals. Portions of the highway have guard rails and k-rail type barriers. Within the LCG landslide complex, a multitude of retaining walls are located on various portions of both the upslope and downslope sides of the highway; these wall types are soldier pile with lagging or soldier pile lagged walls with tiebacks.

3.6.1.2 Site Topography

Alternative X generally follows the same route as the existing U.S. 101 route through the project area. Alternative X is positioned on the west flank of a northwest-southeast trending ridge that forms the dominant topographic feature of the project. The ridge is generally bound by the Wilson Creek drainage on the east and the Pacific Ocean on the west. Slopes along the highway corridor range from moderate to extremely steep (Willis, 2000). Ground elevations along the existing highway range within the LCG landslide complex from approximately 540 feet to 960 feet.

Between Wilson Creek and approximately PM 14.45, Alternative X crosses an area of the landslide complex interpreted as an active earthflow. The surface topography is characterized by gently rolling irregular slopes. On the eastern side of the highway, the slopes average approximately 4 horizontal to 1 vertical (4H:1V). On the western side of the highway, as the earthflow approaches the coastline, the slopes get steeper than 2H:1V.

North of the active earthflow, between approximately PM 14.45 and PM 15.57, the topography becomes steeper as the landslide complex is dominated by a series of massive, interconnected bedrock landslides known as the LCG landslide complex. The slopes in this area average approximately 1½H:1V. The eastern edge of this area is bordered by a zone of steep scarps separating extensional blocks with gentle slopes or depressions. The western side of this slide complex is bordered by steeper slopes descending to the Pacific Ocean. The steepest slopes are along the sea cliffs, with areas as steep as approximately ½H:1V average slope.

From PM 15.57, the highway extends northeast away from the coastline and continues along more gently sloped, rolling topography.

3.6.1.3 Site Surface Water and Drainage Conditions

From the ridgeline above the Alternative X alignment, surface water generally flows in a westward direction toward the Pacific Ocean. No named drainage features, such as a stream, creek, or river, are mapped within the project limits. The drainage conditions are interconnected with the geomorphic expression of the landslide complex as established drainages have been unable to develop due to the landslide movement. The sea cliffs on the western side of highway are scared by debris slides which create steep drainage pathways and erosional rills.

3.6.1.4 Significant Natural Site Features

Most of the west flank of the ridge and all of U.S. 101 are located within the Del Norte Coast Redwoods State Park and Redwood National Park. The predominant natural site features are redwood trees and other vegetation.

3.6.1.5 Site Land Use History

The area has historically been used as a corridor between Klamath and Crescent City. The 1929 and subsequent topographic maps show the highway generally in its current alignment. Former structures along U.S. 101, approximately ¾ to 1 mile north of Wilson Creek Bridge, were visible in a 1983 aerial photograph and along Rudisill Road. Redwood National Park was established in

1958 and the Del Norte Coast Redwoods State Park was established in 1927, providing public access to redwood forest and coast in the project area. A stretch of California's Coastal Trail is located east and runs generally parallel to U.S. 101.

3.6.1.6 Performance of Existing Natural and Engineered Site Slopes

The performance of existing natural and engineered site slopes along Alternative X has generally been poor. As discussed previously, the entire slope between the ridge top and the ocean between PM 14.45 and PM 15.57 is the very active large deep-seated LCG landslide complex. Between the LCG landslide complex and Wilson Creek, Alternative X crosses the much slower moving, but active earthflow area.

The existing natural and engineered slopes (cut slopes on the inboard side of the highway) within the LCG landslide complex have failed repeatedly, resulting in creeping lateral movements, vertical settlement, upslope rockslides, and severe damage to the roadway and retaining structures. These damaging movements have been ongoing since the highway was constructed.

Within the earthflow area to the south, the existing natural and engineered slopes have generally performed adequately requiring much less repair than within the LCG landslide complex. This is due to the much gentler topography and the generally slow-moving nature of the earthflow.

3.6.1.7 Historical Maintenance Issues and Emergency Repairs

Repeated and ongoing landslide activity has impacted the current alignment since shortly after the route was completed in the 1920s (Caltrans, 2018). Even during construction, many slipouts and slides occurred, delaying construction (Caltrans, 2015). According to the 2015 Engineered Feasibility Study, the landslide has moved the highway over 50 feet horizontally from the 1937 alignment. Between 1981 and 2012, a little over $36 million was spent on maintenance and repair projects ranging from filling and leveling scarps in the roadway surface with pavement to construction of soldier pile retaining walls.

In 1972, the landslide destroyed the roadway and two motorists lost their lives. In 2011, three slipouts resulted in closure of the southbound shoulder and requiring resurfacing the highway and an extension of an existing retaining wall. In 2012, a storm-related slipout required an emergency soil nail wall. Slide movement between 2012 and 2015 resulted in visible damage to retaining walls at the NLCG and SLCG slide interface, resulting in Emergency Opening projects (Caltrans, 2015).

Landslides in February 2021 completely closed the highway at the north end of the project area for five days. Emergency repair work required nearly two years to complete and included slide debris removal and installation of an anchored cable-net drapery system and retaining walls. The repairs necessitated partial closures of U.S. 101 in the affected area for the duration of the work.

Since the 2021 slide, smaller slides have partially or completely closed the highway and required earthwork but no structural mitigations.

Table 12 below lists previous projects on LCG.

Table 12. Previous Last Chance Grade Projects

Project Description [1]	Project Location (PM)	Year of Project
Storm Damage Repair	4.6 to 36.0	1957
Storm Damage Repair	15.3	1972
Storm Damage Repair	14.41 to 14.52	1985
Construction of an Anchored Retaining Wall	14.41 to 14.52	1987
Repair of Anchored Wall	14.5	1997
Slipout and Washout Repair	15.2 to 22.8	1998
Construction of Last Chance Grade Retaining wall	15.5	1999
Construction of Wilson Creek Retaining Wall	14.6	2000
Seal Cracks in Roadway	9.4 to 15.6	1999
Placement of Open Grade Asphalt Concrete	15 to 15.4	2000
Drainage Revision	12.7 to 12.9	2002
Reconstruction of the Roadway and placement of Open Grade Friction Course	14.4 to 14.8	2009
Construction of Retaining Walls	15.0 to 15.4	2010
Three Slipout Repairs	15.0 and 15.27	2012
Emergency Soil Nail Wall	15.27	2012
Rubberized Hot Mix Asphalt	12.7 to 15.5	2012
Construction of Soldier Pile Wall	15.3	2013
Anchored cable-net drapery system and SPGA walls [2]	15.48	2023

(1) Source: Caltrans Engineering Feasibility Study, June 25, 2015, Table 3.
(2) Source: Caltrans email to HNTB dated September 13, 2023.

3.6.2 Site Landslide Conditions – Alternative X

There are four separate landslide complexes along this alignment, as described in Section 3.1.3.1. Three of these are within the Broken Formation of the Franciscan Complex and include (from north to south) the NLCG, SLCG, and WC. These three landslide complexes are composed primarily of sandstone of the Broken Formation with lesser amounts of argillite interbeds.

The fourth and southern-most landslide complex within the project area is the EF that has developed within the Melange unit of the Franciscan Complex. This body is composed primarily of intensely sheared argillite with irregularly positioned sandstone blocks of sizes measuring up to tens of feet across. The shearing within this body is the result of both tectonic movement as well as subsequent landslide mobilization. These multiple episodes of shearing activity have resulted in a heterogenous to chaotic structure with resulting argillite fragments generally ranging from sand particle to several inches across with zones that include silt and clay fines.

The argillite interbeds within the sandstone blocks of the NLCG, SLCG, and WC are generally less sheared than what is observed in the EF and present a more shattered texture with harder and larger clasts size and less fines. A more detailed description of these landslide units and their applicability to the slope stability analyses is presented in Section 3.4.

3.6.3 Site Subsurface Conditions – Alternative X

3.6.3.1 Existing Underground Structures, Facilities, and Utilities

As-built plans for most Caltrans structures in the vicinity of Alternative X are available from Caltrans. Plans and/or details for in-progress or recently completed Caltrans repair structures along the existing highway alignment dated between 2015 and 2021 were provided for review.

Existing underground structures along Alternative X consist of current roadway stability structures. Retaining walls can be found continuously between PM 14.9 and 15.5 and include the following:

- A 450-foot-long anchored soldier pile wall with tiebacks constructed on the west side of the highway at PM 14.89 (Caltrans, 2018).
- An approximately 212-foot-long, 25-foot-high soil nail wall constructed on the west side of the highway between PM 15.10 to 15.14 (Caltrans, 2018).
- An approximately 45-foot-long retaining wall (RW 5) overlapped by an approximately 140-foot-long anchored soldier pile wall with tiebacks (RW 5A) constructed on the west side of the highway at PM 15.45. Wall RW 5A is partially overlapped to the south by an approximately 625-foot-long anchored soldier pile wall with tiebacks (RW 5B) (Caltrans, 2020a).
- Across from Wall RW 5B on the east side of the highway are an approximately 217-foot-long concrete pilaster wall and tiebacks (RW 3) at PM 15.06 (Caltrans, 2020a and 2018c) and an approximately 133-foot-long retaining wall (RW 4) approximately 85 feet north of Wall RW 3.

No live underground utilities are believed to be present along the Alternative X alignment between PM 14.3 and PM 15.9. SI casing, VWPs, and standpipe piezometers are located within and adjacent to the current roadway section. A pneumatic hammer and drill string were also found at approximate PM 14.75.

3.6.3.2 Distribution of Site Soil/Rock Units

Alternative X would transect the four landslide complexes present at the site. The lateral distribution of landslide deposits for Alternative X is presented below.

- NLCG (incorporates local colluvial deposits, Qc, and landslide deposits derived from Broken Formation, map symbol Qlsd-bf): Station 501+00 to Station 506+70 (north end of Alternative X alignment)
- SLCG (incorporates local colluvial deposits, Qc, and landslide deposits derived from Broken Formation, map symbol Qlsd-bf): Station 482+00 to Station 501+00
- WC (incorporates local colluvial deposits, Qc, and landslide deposits derived from Broken Formation, map symbol Qlsd-bf): Station 453+00 to Station 482+00
- EF (incorporates local colluvial deposits, Qc, and landslide deposits derived from

Melange, map symbol Qlsd-m): Station 450+00 (south end of Alternative X alignment) to Station 453+00

The grading and improvements proposed for the Alternative X alignment are situated entirely within each respective landslide complex mass and would not penetrate the basal failure surfaces at any point.

Geotechnical properties of project area soil/rock units are discussed in Section 3.4. Ground conditions for tunneling for the Alternative X underground drainage system are discussed in Section 3.6.3.3.

3.6.3.3 Site Tunneling Conditions - Alternative X Underground Drainage System

- *Definition of Tunnel Reaches*

Three reaches were defined for each of the Alternative X drainage gallery tunnels and shafts. Limits were based on geology and ground conditions, proposed structures, and anticipated construction method and were defined as follows:

- Limits of Reach 1 were defined as the limits of the 30-foot diameter drainage shafts.
- Limits of Reach 2 were defined as the length of the TBM-bored drainage gallery tunnels to be constructed through GC III Franciscan Complex Melange, starting from the breakout at the north side of the shaft.
- Limits of Reach 3 were defined as the length of the TBM-bored drainage gallery tunnels to be constructed in Franciscan Complex Broken Formation, from the end of Reach 2 to their endpoints at the north end of the system.

The contact between the Melange and the Broken Formation was inferred from available subsurface information and may be revised as additional information becomes available. For purposes of estimating ground class distributions for tunneling, the contact was assumed to be vertical. All three drainage gallery tunnels would be constructed below the groundwater table.

- *Distribution of Site Ground Classes*

The estimated distribution of tunneling ground classes for Alternative X underground drainage system was estimated based on projections of available subsurface data, geologic mapping, and geologic interpretation as shown in Plates 9a through 9d. Table 13 shows the estimated distribution of Ground Classes within each reach for the high-level, mid-level, and low-level drainage gallery tunnels. It also summarizes percentages of ground classes for all proposed Alternative X excavation and also for just the three drainage shafts. The table also shows the estimated distribution of Ground Class Groups for all proposed Alternative X excavation and for just shaft excavation. All percentages shown are by volume, and assumptions and dimensions for the ground class estimation are included in Table 13.

As shown, it is estimated that about 87 percent of the excavation for the Alternative X underground drainage system would be in GC II Broken Formation. For the drainage shafts, it is estimated that slightly less than half of the excavation would be in GC V earthflow, and slightly more than half would be in GC III Melange.

Although based on the limited available subsurface data, the distributions and relative proportions of Ground Classes shown in Table 13 and discussed in the following sections are suggested for initial planning purposes. They will be refined as additional data become available.

Table 13. Estimated Ground Class Distribution for Alternative X Underground Drainage System

Ground Classes												Ground Class Groups		
Ground Class	Percent Volume for Reach									Percent Volume for All Excavation	Percent Volume for Shaft Excavations	Ground Class Group	Percent Volume for All Excavation	Percent Volume for Shaft Excavations
	High-Level Gallery			Mid-Level Gallery			Low-Level Gallery							
	1 Shaft	2 Breakout Zone	3 TBM Tunnel	1 Shaft	2 Breakout Zone	3 TBM Tunnel	1 Shaft	2 Breakout Zone	3 TBM Tunnel					
Fill/Colluvium/Alluvium	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	Overburden	0.0	0.0
V (Qlsd-m)	47.9	0.0	0.0	54.8	0.0	0.0	38.1	0.0	0.0	6.3	47.0	Landslide Deposits	6.3	47.0
IV (Qlsd-bf)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0			
III (KJFm)	52.1	100.0	0.0	45.2	100.0	0.0	61.9	100.0	0.0	6.8	53.0	Franciscan Complex, Melange	6.8	53.0
II (KJFbf-2)	0.0	0.0	100.0	0.0	0.0	100.0	0.0	0.0	100.0	86.8	0.0	Franciscan Complex, Broken Formation	86.8	0.0
I (KJFbf-1)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0			

Notes and Assumptions

1. Alignment and geologic profile are current as of 10/26/23.
2. Assume Tunnel Horizon is 12 feet.
3. Assume shaft diameters are 30 feet.
4. Assume Reach 3 is all GC II Broken Formation.
5. Assume contact between Melange and Broken Formation is vertical for purposes of this estimation.
6. Depth of high-level shaft = 240 feet.
7. Depth of mid-level shaft = 210 feet.
8. Depth of low-level shaft = 210 feet.
9. Length of high-level TBM tunnel = 6,700 feet.
10. Length of mid-level TBM tunnel=6,800 feet.
11. Length of low-level TBM tunnel = 7,200 feet.
12. Earthflow thickness at high-level shaft = 115 feet.
13. Earthflow thickness at mid-level shaft = 115 feet.
14. Earthflow thickness at low-level shaft = 80 feet.
15. Length of high-level TBM tunnel in Melange = 640 feet (breakout zone).
16. Length of mid-level TBM tunnel in Melange = 320 feet (breakout zone).
17. Length of low-level TBM tunnel in Melange = 330 feet (breakout zone).

- *Site Tunneling Conditions by Reach*

<u>Reach 1 (Drainage Shafts)</u>

Reach 1 comprises the 30-foot diameter drainage shaft for each of the three drainage gallery tunnels. The uppermost material to be excavated at each shaft would be GC V earthflow. Earthflow thickness is estimated to be 115 feet at the high- and mid-level shafts and 80 feet at the low-level shaft.

Beneath the earthflow material the excavation would encounter GC III Franciscan Complex Melange to the bottom of the shaft. Melange thickness is to be excavated is estimated to be 125 feet at the high-level shaft, 95 feet at the mid-level shaft, and 130 feet at the low-level shaft. As shown in Table 13, earthflow would constitute slightly less than half (47 percent) of all excavations for the shafts, and Melange would constitute slightly more than half (53 percent).

The earthflow through which the shafts would be constructed is active, and earth movements are anticipated during and after construction.

High (>500 gallons per minute) groundwater inflows are not anticipated in the shaft excavations. Groundwater inflow estimates will be re-evaluated after additional explorations have been completed.

Geotechnical properties of the Melange and the earthflow deposits are discussed in Sections 3.4 and 3.5.

<u>Reach 2 (Breakout Zone in Melange)</u>

Reach 2 comprises the breakout zone of the TBM tunnels from the drainage shafts. Reach 2 would be constructed entirely in GC III Franciscan Complex Melange. The length of this reach would be 640 feet for the high-level drainage gallery tunnel, 320 feet for the mid-level drainage gallery tunnel, and 330 feet for the low-level drainage gallery tunnel.

High groundwater inflows (>500 gallons per minute) are not anticipated in excavations in Reach 2 because weathering of the argillite of the Melange is likely to have effectively reduced fracture flow. However, open fractures could produce moderate inflows.

Geotechnical properties of the Melange are discussed in Sections 3.4 and 3.5.

<u>Reach 3 (TBM Tunneling in GC II Broken Formation)</u>

Reach 3 comprises the majority of the length of the TBM-bored drainage gallery tunnels. The tunnels would be bored primarily with full-face excavation in GC II Broken Formation. However, some mixed-ground excavation could be necessary at the Reach 2 – Reach 3 boundary.

The southern limit for Reach 3 is the contact between the Melange and Broken Formation. The location, orientation, and nature of this contact along the Alternative X drainage gallery tunnels are not known. It could be an abrupt boundary or a gradational or sheared zone several hundred feet wide. It is inferred to be steeply dipping.

Open fractures in Broken Formation sandstone could be capable of producing high (>500 gallons per minute per 1,000 feet) flush flows if not treated. Additional site exploration and testing would help to confirm groundwater conditions.

Geotechnical properties of the Broken Formation are discussed in Sections 3.4 and 3.5.

3.6.4 Site Groundwater Conditions – Alternative X

Alternative X is impacted by all four landslide complexes. Observed influences of groundwater on slope stability are discussed relative to each of these complexes below.

3.6.4.1 North Last Chance Grade Complex (NLCG)

The subsurface investigation of the NLCG included advancement of Borings RC-19-003, RC-20-013, and RC-20-017. Inclinometer casing and VWPs were installed in all three borings. Groundwater data from the single VWPs installed in Borings RC-19-003 and RC-20-013 indicate an apparent piezometric level of 11.8 feet BGS and 82.9 feet BGS at their shallowest, respectively. Groundwater data from the VWPs within Boring RC-20-017 (all of which were installed below the apparent basal failure surface/zone) exhibit minimal fluctuation over time, and a shallowest apparent piezometric level at a depth of approximately 138.1 feet BGS.

Boring RC-19-003 is located east of the landslide complex. No basal failure surface was logged within the boring, and inclinometer surveys indicate no apparent deformation or displacement. The boring records and cross-sectional analysis within the NLCG complex suggest the basal failure surface/zone is approximately 16 feet BGS in Boring RC-20-013 and approximately 82 feet BGS in Boring RC-20-017. The inclinometer surveys within the two borings indicate no apparent deformation or displacement, and groundwater elevations as measured in Borings RC-20-013 and RC-20-017 are below the approximate basal failure surface/zone.

3.6.4.2 South Last Chance Grade Complex (SLCG)

The subsurface investigation of the SLCG included advancement of Borings P-20-002, RC-20-011, and RC-20-015. Only VWPs were installed in Boring P-20-002, while VWPs and inclinometer casing were installed in Borings RC-20-011 and RC-20-015. Groundwater data from P-20-002 indicate an apparent piezometric level at a depth of 125.5 feet BGS at its shallowest. Groundwater data from the VWPs within Boring RC-20-011 recorded an apparent minimum piezometric level at a depth of approximately 165.9 feet from the VWP installed below the landslide mass and 144.1 feet from within the landslide mass. Data from VWPs indicate significant increases in elevation in late winter to early spring. The elevated groundwater levels that slowly dissipate through autumn are also coincident with significant rainfall and storm activity at the site during the same period.

Boring records and cross-sectional analysis within the complex suggest the basal failure surface/zone is approximately 261 feet BGS in Boring RC-20-011 and approximately 161 feet BGS in Boring RC-20-015 (no boring record was produced for Boring P-20-002 due to the drilling system utilized and insufficient drill cuttings returned to the surface). Inclinometer surveys of Boring RC-20-011 indicate no apparent deformation or displacement during the late winter and early spring of 2021. The profile for Boring RC-20-015, however, exhibits displacement along a failure surface at approximately 97 feet BGS. Rate of displacement from early February through March 2021 is relatively linear. Surveys acquired in April and beyond indicate a significant decrease in displacement rate, and in September, a second failure surface exhibits displacement at a depth of approximately 161 feet.

While the elevated groundwater levels in Boring RC-20-015 recorded during late winter to early spring are below the failure surface on which displacement was occurring within the inclinometer at those times, the concurrent groundwater dissipation and decreased displacement rate through autumn would suggest an overall influence on the stability of the complex mass. Given increases in groundwater elevation also coincided with storm activity, the effects of coastal erosion at the toe due to increased wave action should not be precluded during this period, and this phenomenon should be evaluated in future investigations.

3.6.4.3 Wilson Creek Complex (WC)

The subsurface investigation of the WC included advancement of Borings P-19-007, RC-20-005, D-20-009, RC-20-014, and RC-20-016. Only VWPs were installed in Boring P-19-007, while VWPs and inclinometer casing were installed in the remaining borings. Groundwater data from the five (5) VWP locations indicate apparent piezometric levels at depths from 39.4 to 147.8 feet BGS at their shallowest. Groundwater may be confined at the deepest VWP in D-20-009 where an upward gradient is indicated between the two deepest VWPs. Data analysis indicates locally elevated groundwater levels, peaking generally in late winter to spring and slowly dissipating through autumn. This interval is coincident with significant rainfall and storm activity at the site slightly preceding or during the same period.

Boring records and cross-sectional analysis within the WC suggest the basal failure surface/zones within Borings RC-20-005, D-20-009, RC-20-014, and RC-20-016 are at approximate depths of 157 feet, 270 feet, 77 feet, and 287 feet, respectively. No boring record was produced for Boring P-19-007 due to the drilling system utilized and poor recovery. Inclinometer surveys of Borings D-20-009 and RC-20-016 exhibit displacement along multiple planes through the depth of the casing.

The VWP data indicate groundwater may be confined within and below the WC mass, at least locally and episodically. Groundwater elevations in Borings RC-20-005 and RC-20-014 indicated by VWPs installed below the landslide mass do not exceed the basal failure surface at those locations. Increases in groundwater elevation at Boring RC-20-005 within the mass peaks in late winter to early spring. Data from VWPs within Borings D-20-009 and RC-20-016 indicates significant groundwater elevation increase, particularly at depth, also peaking in late winter to early spring. The subsequent groundwater dissipation and decreased displacement rate (from inclinometer data) suggest influence on the stability of the complex mass by groundwater. Given the increases in groundwater elevation also coincided with storm activity, the effects of coastal erosion at the toe due to increased wave action should not be precluded during these periods.

3.6.4.4 Large Earthflow Complex (EF)

The subsurface investigation of the EF included advancement of Borings RC-18-001, RC-19-004, RC-20-006, D-20-010, RC-20-019, RC-20-020, and RC-21-001. VWPs and inclinometer casing were installed in the seven borings. Groundwater data from the installations indicate apparent piezometric levels at depths from 4.4 to 58.2 feet BGS at their shallowest, except Boring D-20-010. Boring D-20-010 had apparent levels above the existing ground surface, suggesting a confined/artesian condition. Data analysis indicates locally slightly elevated groundwater levels (Borings RC-20-019 and RC-20-020), peaking in spring and slowly dissipating through autumn. This interval is coincident with significant rainfall and storm activity at the site slightly preceding or during the same period. However, groundwater level fluctuation in the installations was limited to less than approximately 3 to 5 feet vertically.

Boring records, inclinometer surveys and cross-sectional analysis within the EF suggest the basal failure surface/zones within Borings RC-18-001, RC-19-004, RC-20-006, D-20-010, RC-20-019, RC-20-020, and RC-21-001 are at approximate depths of 67 feet, 49 feet, 80 feet, 67 feet, 143 feet, 62 feet, and 98 feet, respectively. Inclinometer data collected at RC-21-001 in February 2023 suggests an increased rate of movement relative to prior readings at the failure zone between approximately 90 to 96 feet. Displacement plots within this zone indicate a rate of 1.17 inches/year was measured with total movement to date of 1.12 inches. Inclinometer surveys of Borings RC-20-019 and RC-20-020 do not indicate casing deformation or displacement.

The collected VWP data from the installations indicates only minor groundwater elevation fluctuation within and below the EF landslide mass throughout the year. While it is apparent the mass moves at varied rates locally, in general, the constant groundwater presence above and below the basal failure surface/zone likely results in relatively linear creep rate of the mass.

3.6.5 Preliminary Seismic Hazard Evaluation – Alternative X

Preliminary seismic hazard evaluation for Alternative X is described in this section. It should be noted that all recommendations provided in this section shall be further confirmed or revised when site-specific data from future field investigations become available and more detailed analyses are performed.

3.6.5.1 Site Seismic Parameters

Site seismic parameters were evaluated for the structures along the alignment. According to the current plans and drawings, six retaining walls are proposed along Alternative X alignment. Some of the adjacent retaining walls were considered as one site and the same set of seismic/ground motion parameters was recommended for both structures. As a result, five site locations were selected for seismic hazard evaluation.

- *Sites Where Shear-Wave Velocity V_{S30} Values are Evaluated*

Table 14 lists the geospatial coordinates (latitude/longitude) of the five selected sites along Alternative X alignment, where time-averaged shear-wave velocity (V_{S30}) for the top 30 meters (100 feet) of earth material was evaluated. The geospatial site coordinates were estimated according to Google Maps, using the current project plans.

Table 14. Preliminary Site Seismic Parameters (Alternative X)

Structure(s)	**RW 6**	**RW 7A-1**	**RW 7A-2**	**RW 7A-3**	**RW 7B**	**RW 7C**
Station Range (1)	'X' 479+00 to 'X' 481+00	'X' 455+00 to 'X' 468+65	'X' 470+50 to 'X' 479+55	'X' 480+30 to 'X' 513+00	'X' 499+52 to 'X' 503+51	'X' 499+77 to 'X' 503+02
Reference Boring(s) (2)	RC-20-014 RC-20-016	R-19-001 RC-20-005 RC-20-006	RC-20-005	R-19-003 RC-20-005 RC-20-011 RC-20-013 RC-20-017	R-19-003 RC 20-01 RC 20-013 RC 20-017	
Site Geospatial Coordinates (3) (latitude/ longitude)	41.6328°, -124.1146°	41.6263°, -124.1124°	41.6309°, -124.1132°	41.6393°, -124.1152°	41.6378°, -124.1150°	
V_{S30} (m/s) (4)	310	280	310	310	320	

<u>Notes:</u>

(1) Based on *X_WALL-Review set_20221104.pdf* plan sheet.

(2) Based on *2022-0708_SUB032_Prelim-Geotech-Data-Report_Final-2.pdf* (Caltrans, 2022a).

(3) Estimated per Google Maps and the current Geometric Approval drawings.

(4) For a conservative approach, a lower-bound V_{S30} value along the alignment of Retaining Walls was adopted for generating its ARS curve.

- *Time-Averaged Shear-Wave Velocity V_{S30} for Top 30 m of Earth Materials*

Shear wave velocity for the alignment was estimated at limited locations using direct geophysical surveys (seismic refraction survey). Due to the limited number of these measurements and their distance from the structure sites, the V_{S30} values were also estimated in accordance with the Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations (Caltrans, 2012 and 2021b), based on the correlation with standard penetration test (SPT) blow counts recorded in the nearby borings. The estimated V_{S30} values from different explorations for Alternative X are listed in Table 15. Details are provided in Appendix B-1.

After comparison with the existing limited P- and S-wave suspension logging and seismic refraction measurements, it was observed that the 2021 Caltrans correlations tend to yield a much lower value of V_{S30} than shear wave velocity estimates from seismic refraction lines. The 2012 correlations result in V_{s30} estimates that better match the values form seismic refraction lines. Therefore, for preliminary evaluations, the V_{S30} values estimated from SPT blow counts using 2012 Caltrans procedures have been used for structure sites. The V_{s30} values will be verified once site-specific geotechnical investigation is performed for each structure.

Table 15. Estimated V_{S30} from Geotechnical/Geophysical Explorations (Alternative X)

Type of Exploration	**Exploration ID**	**Estimated V_{S30} (m/s)**		
		From Field Blow Counts		**From Seismic Refraction**
		Caltrans 2012 Method (1)	**Caltrans 2021 Method (2)**	
Soil Boring	RC-19-001	284	NA	NA
	RC-19-003	397	NA	NA
	RC-20-006	270	202	NA
	RC-20-005	353	326	NA
	RC-20-011	319	270	NA
	RC-20-013	309	264	NA
	RC-20-014	285	246	NA
	RC-20-016	334	294	NA
	RC-20-017	361	324	NA
Seismic Refraction Line	SL-11	NA	NA	594
	SL-12	NA	NA	565
	SL-13A	NA	NA	427
	SL-13B	NA	NA	338
	SL-42	NA	NA	456
	SL-43	NA	NA	670

Notes:
(1) V_{S30} calculated as per Caltrans (2012).
(2) V_{S30} calculated as per Caltrans (2021b).

3.6.5.2 Ground Motion Parameters

Ground motion parameters were evaluated in general accordance with Caltrans Geotechnical Manual - Design Acceleration Response Spectrum (ARS) module (Caltrans, 2020a). According to this module, unless specified otherwise in a Project-Specific Seismic Design Criteria, Caltrans current practice is to use the Safety Evaluation Earthquake (SEE) design ARS developed per Caltrans Seismic Design Criteria Version 2.0 (SDC 2.0) (2019a) to characterize design ground motions for earth retaining systems (ERSs), embankments, slopes, sign structures and other appurtenant highway facilities. Details are provided in Appendix B-2.

Based on the procedures described in SDC 2.0 and October 2019 Interim Revisions to SDC 2.0 (2019b), the preliminary ARS curves for SEE with a 975-year Return Period at the five selected sites were determined using the Caltrans ARS Online V3.0.2 (2023c) website. The site coordinates and V_{S30} values as shown in Table 14 were used in these evaluations. The resulting preliminary ground motions parameters and ARS are provided in Table 14 Table 16. The soils for various retaining wall sites are all identified as "Class S1", per Sections 6.1 and 6.2.3 of the Caltrans SDC 2.0 (2019a).

- *Horizontal Peak Ground Acceleration (HPGA)*

The preliminary Horizontal Peak Ground Acceleration (HPGA) values for the five selected sites are summarized in Table 16. The evaluated HPGA values for the five sites are nearly identical, varying from 0.86g to 0.88g.

Table 16. Preliminary Ground Motion Parameters (Alternative X)

Structure(s)		RW 6	RW 7A-1	RW 7A-2	RW 7A-3	RW 7B / RW 7C
Site Geospatial Coordinates (latitude/longitude)		41.6328°, -124.1146°	41.6263°, -124.1124°	41.6309°, -124.1132°	41.6393°, -124.1152°	41.6378°, -124.1150°
Horizontal Spectral Acceleration (g)	PGA	0.870	0.880	0.870	0.870	0.860
	0.1 sec	1.430	1.400	1.430	1.430	1.430
	0.2 sec	1.770	1.730	1.770	1.760	1.770
	0.3 sec	1.880	1.890	1.880	1.870	1.860
	0.5 sec	1.680	1.760	1.680	1.670	1.650
	0.75 sec	1.420	1.510	1.420	1.420	1.390
	1 sec	1.180	1.250	1.180	1.180	1.150
	2 sec	0.610	0.660	0.610	0.610	0.590
	3 sec	0.370	0.400	0.370	0.370	0.360
	4 sec	0.250	0.270	0.250	0.250	0.240
	5 sec	0.170	0.180	0.170	0.170	0.170
Mean Earthquake Moment Magnitude		8.66	8.65	8.66	8.66	8.66
Mean Site to Fault Source Distance for S_a at 1 second (km)		20.0	20.0	20.0	20.0	20.0
Horizontal Seismic Coefficient		0.290	0.293	0.290	0.290	0.287

- *Mean Earthquake Moment Magnitude (M)*

The Caltrans ARS Online V3.0.2 (2021c) also provides the Mean Earthquake Moment Magnitude (M) based on the 2014 USGS hazard deaggregation analysis for the HPGA scenario. Table 16 also includes these preliminary M values for the five selected sites. The evaluated values for all sites are also quite close, varying from M8.65 to M8.66.

- *Mean Site to Fault Source Distance*

The Caltrans ARS Online V3.0.2 (2021c) also provides the Mean Site to Fault Source Distance based on the 2014 USGS hazard deaggregation analysis for the spectral acceleration at 1 second. Table 16 also lists the preliminary mean distance, which is 20.0 kilometers for the five selected sites.

3.6.5.3 Parameters for Seismic Slope Stability Analysis

According to the Caltrans Geotechnical Manual – Landslides (Caltrans, 2020b) and Caltrans Geotechnical Manual - Embankments (Caltrans, 2014), a horizontal seismic coefficient for seismic slope stability analysis equal to one-third of the horizontal peak horizontal acceleration at the site can be used for preliminary seismic slope stability evaluations. These horizontal seismic coefficients for the five selected site locations are tabulated in Table 16. Alternatively, a displacement-based seismic slope stability approach, similar to the one recommended in Caltrans Geotechnical Manual – Liquefaction-Induced Lateral Spreading module (Caltrans, 2020c), may be used to evaluate the seismic stability of landslides.

3.6.5.4 Fault Rupture

In general terms, an earthquake is caused when strain energy in rocks is suddenly released by movement along a plane of weakness. In some cases, fault movement propagates upward through the subsurface materials and causes displacement at the ground surface. Surface rupture usually occurs along traces of known or potentially active faults, although many historic events have occurred on faults not previously known to be active.

Fault rupture hazard was evaluated in accordance with Caltrans Geotechnical Manual – Fault Rupture module (Caltrans, 2017) and Caltrans Memo to Designers (MTD) 20-10 (Caltrans, 2013). MTD 20-10 requires a Surface Fault Rupture Displacement Hazard Analysis (SFRDHA) where any portion of the structure is located:

- Within an APEFZ, as defined by the CGS.
- Within 1,000 feet of an unzoned fault (i.e., not located in an APEFZ) that is Holocene (11,000 years) or younger in age.

Caltrans uses the Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) model (USGS, 2013) for its fault database. This model does not separate Holocene aged faults (11,000 years) from Holocene-Latest Pleistocene (active within the last 15,000 years). Accordingly, structures located near faults that are Holocene-Latest Pleistocene age or younger must be evaluated for potential fault rupture hazard.

The CGS establishes criteria for faults as active, potentially active, or inactive. Active faults are those that show evidence of surface displacement within the last 11,000 years (Holocene age). Potentially active faults are those that demonstrate displacement within the past 1.6 million years (Quaternary age). Faults showing no evidence of displacement within the last 1.6 million years may be considered inactive for most structures, except for critical or certain life structures. In 1972, the Alquist-Priolo Special Studies Zone Act (now known as the Alquist-Priolo Earthquake Fault Zone Act, 1994) was passed into law which requires studies be performed when within 500 feet of active or potentially active faults. The Alquist-Priolo Earthquake Fault Zone (APEFZ) designates “active” and “potentially active” faults utilizing the same age criteria as that used by the CGS. However, the established policy is to zone active faults and only those potentially active faults that have a relatively high potential for ground rupture.

The alignment of Alternative X does not transverse within 1,000 feet of any active faults as delineated by the APEFZ (CGS, 2007) or UCERF3 model. Therefore, based on the current Caltrans criteria, the potential for surface ground rupture along the subject alternative alignment is negligible.

3.6.5.5 Liquefaction

Liquefaction is a phenomenon whereby saturated granular soils lose their inherent shear strength due to increased pore water pressures, which may be induced by cyclic loading such as that caused by an earthquake. Low relative density granular soils, shallow groundwater, and long duration and high acceleration seismic shaking are some of the factors favorable to cause liquefaction. Liquefaction is generally considered possible when the depth to groundwater is within about 50 feet from the ground surface.

Liquefaction hazard was evaluated in accordance with Caltrans Geotechnical Manual – Liquefaction module (Caltrans, 2020d) and using SPT blow counts.

Preliminary liquefaction potential analysis was performed using the procedures outlined by Youd and Idriss (2001), the blow counts and measured groundwater depths of the existing nearby borings, and the preliminary ground motion parameters listed in Table 17. The seismically induced settlements were estimated using the empirical method proposed by Tokimatsu and Seed (1987). The preliminary results are summarized in Table 17. Due to the presence of deep groundwater, no liquefiable layers are identified. The larger seismically induced settlements at some locations are mainly derived from dry sand settlements above groundwater and at shallow depths of approximately 5 to 15 feet. If the proposed footing bottom elevation of a retaining wall is placed deeper than these depths, seismically induced settlements affecting the structure will be lower.

Table 17. Summary of Preliminary Liquefaction Analysis Results (Alternative X)

Structure	Reference Boring(s)	Surveyed Ground Surface Elevation (feet)	Measured Groundwater Elevation (1) (feet)	Moment Magnitude	HPGA (g)	Liquefiable Layer Elevations (feet)	Seismically Induced Settlement (inch)
RW 7A-1	RC-19-001	+538.8	+478.8	8.65	0.88	None	0.7
	RC-20-006	+619.3	+559.3	8.65	0.87	None	11.1 (2)
RW 7A-1 **RW 7A-2** **RW 7A-3**	RC-20-005	+859.1	+715.2	8.65	0.87	None	< 0.1
RW 7A-3	RC-19-003	+840.5	+830.5	8.67	0.85	None	<0.1
	RC-20-013	+830.5	+697.5	8.67	0.85	None	0.4
	RC-20-014	+805.1	+639.1	8.65	0.87	None	< 0.1
	RC-20-016	+674.4	+538.4	8.65	0.87	None	4.9 (2)
	RC-20-017	+829.4	+679.4	8.67	0.85	None	1.7 (2)
RW-6 **RW-7A-3** **RW-7B** **RW-7C**	RC-20-011	+698.5	+554.5	8.66	0.87	None	4.3 (2)

Notes:

(1) Groundwater elevation (shallowest measured) extracted from Final Preliminary Geotechnical Data Report (Caltrans, 2022a).

(2) Mainly dry-sand settlement.

3.6.5.6 Liquefaction-Induced Lateral Spreading

Liquefaction-induced lateral spreading hazard shall be evaluated in accordance with Caltrans Geotechnical Manual – Liquefaction-Induced Lateral Spreading module (Caltrans, 2020c). Based on the current information, the potential for lateral spreading at structure sites along the Alternative X alignment is low, as these locations are not underlain by soil susceptible to liquefaction.

3.7 Site Geotechnical Conditions – Alternative F

3.7.1 Site Surface Conditions – Alternative F

3.7.1.1 Existing and Proposed Above-Ground Structures, Facilities, and Utilities

Aside from the existing U.S. 101 connecting at the north and south project limits, there are no existing or other proposed projects that would include above-ground structures, facilities, and utilities affecting Alternative F. As-built plans for Caltrans structures in the vicinity of Alternative F are available from Caltrans.

3.7.1.2 Site Topography

The portion of Alternative F that is in tunnel would be located at depth below the northwest-southeast trending ridge that forms the dominant topographic feature of the project.

The South Tunnel Portal and OMC are located on the northeast side of U.S. 101 about ½ mile north and ¼ mile southeast, respectively, of the Rudisill Road turnout. The portal is within the EF just south of its interface with the WC. The surface topography in this area is characterized by gently rolling, irregular slopes. The LiDAR survey shows several flat areas in the location of the former structures described in Section 3.6.1.5. In general, the surface topography in this area elevation near the South Portal approach area is approximately 580 to 700 feet. The ground surface elevation along U.S. 101 near the OMC ranges from approximately 340 to 355 feet.

The North Tunnel Portal would be located on the east side of U.S. 101, where the highway turns and continues to the northeast. The portal would be just east of the main head scarp of the NLCG. At this location, the portal daylights on a north facing slope near the top of a ravine that extends northeastward toward Wilson Creek. The north facing slope is approximately 1½H:1V to 2H:1V. The south side of the valley has several intervening ridges and valleys. The ground elevation near the North Portal approach area is approximately 740 to 910 feet.

3.7.1.3 Site Surface Water and Drainage Conditions

The portion of Alternative F that is in tunnel will be located at depth below any drainage channels that convey surface water.

There are no mapped perennial or intermittent creeks or streams crossing the alignment at the South Portal (USGS, 1997a and 1997b). The North Portal is, however, located on a north facing slope that is part of a series of drainages and ridges. Surface water is anticipated to flow intermittently in the drainage that crosses the alignment immediately north of the portal. Water in this drainage and adjacent drainages generally flows in a northeastern direction that ultimately leads to Wilson Creek to the east.

The South Portal is located within the active earthflow portion of the landslide complex. The drainage conditions are interconnected with the geomorphic expression of the earthflow as established drainage ways have been unable to develop due to the landslide movement. From the South Portal, surface water is anticipated to flow generally in a southwestern direction downslope towards the Pacific Ocean.

3.7.1.4 Significant Natural Site Features

Similar to what is described in Section 3.6.1.4 for Alternative X, the predominant natural site feature are redwood trees and other vegetation. Most of the west flank of the ridge and all of U.S. 101 are located within the Del Norte Coast Redwoods State Park and Redwood National Park.

3.7.1.5 Site Land Use History

As previously discussed in Section 3.6.1.5 for Alternative X, the area has historically been used as a corridor between Klamath and Crescent City. About 1½ miles south of the South Portal, former structures along U.S. 101, approximately ¾ to 1 mile north of Wilson Creek Bridge, were visible in a 1983 aerial photograph and along Rudisill Road. Redwood National Park was established in 1958 and the Del Norte Coast Redwoods State Park was established in 1927, providing public access to redwood forest and coast in the project area. A stretch of California's Coastal Trail is located east and runs generally parallel to U.S. 101.

3.7.1.6 Performance of Existing Natural and Engineered Site Slopes

The South Portal for the Alternative F tunnel would be located within the active earthflow south of the LCG landslide complex. Although in an active earthflow, the performance of existing natural slopes in this area appears to be relatively stable as the earthflow movement is generally slow; the LiDAR comparison from 2011 to 2020 supports this and shows relatively little change in the ground surface elevations or morphology near the South Portal. Slope inclinometer data collected from RC-21-001 indicates that between December 2021 and December 2022 approximately 0.56 inches of total displacement occurred along the failure zone identified at a depth of between 90 and 96 feet (total displacement since baseline reading on February 2, 2021 of approximately 1.15 inches). There are currently no engineered slopes at the South Portal. Willis describes this EF as having been active, but that very rapid large movements are unlikely because of the properties of the Melange bedrock (Willis, 2000).

The North Portal for the Alternative F tunnel would be located on the east side of the ridge, outside of the LCG landslide complex, on a north/northeast facing slope that is not mapped as a landslide, but daylights in a mapped older debris flow scar and about 200 feet west of a mapped dormant landslide. Several dormant landslides are mapped a few hundred feet further north on both the east and west sides of U.S. 101. There are currently no engineered slopes at the North Portal. In general, the slopes at the North Portal appear to have had relatively good performance.

3.7.1.7 Historical Maintenance Issues

No historical maintenance issues are documented in the North or South Portal footprints. None of the maintenance projects on LCG, listed in Table 12, occurred where the North Portal would connect to U.S. 101. Two of the maintenance projects listed in Table 12 were located where the South Portal connects to U.S. 101: Seal Cracks in Roadway (1999) and Rubberized Hot Mix Asphalt (2012).

3.7.2 Site Landslide Conditions – Alternative F

The South Portal approach would be constructed within the active EF just south of its interface with the WC. The nature of this interface is not fully understood and could affect the stability of the approach structure if its condition and orientation are unfavorable. The North Portal approach would be constructed just east of the main head scarp of the NLCG. The North Portal area includes colluvium and dormant debris landslide deposits underlain by Broken Formation.

The ongoing landslide movement could be exacerbated by earthquakes. The project site is located along the CSZ and overlies the interface associated with the subducting crustal plate. This subduction interface is a low angle, east-dipping “megathrust” fault capable of generating great earthquakes of high magnitude (>M8.5).

The overall stability of the Alternative F tunnel alignment would not be affected by the global stability of the LCG landslide complexes if it is sufficiently deep. If the crown of the tunnel can be maintained at least 20 to 40 feet below the basal failure zone, effects of landsliding on the tunnel should be minimal. The tunnel would be subjected to both ground and groundwater pressures and could be subjected to intense seismic ground shaking (M8.57 to M8.67) during its service life.

3.7.3 Site Subsurface Conditions – Alternative F

3.7.3.1 Existing Underground Structures, Facilities, and Utilities

Existing underground structures in the vicinity of Alternative F consist of current roadway stability structures (retaining walls) along U.S. 101. No live or abandoned underground utilities are believed to be present. SI casing and VWPs are located within and adjacent to the current roadway section near where Alternative F joins U.S. 101.

There are no known other proposed projects that would include underground structures or utilities at this time.

3.7.3.2 Site Tunneling Conditions - Alternative F

- *Definition of Tunnel Reaches*

Five reaches were defined for the proposed Alternative F alignment. Reach limits were defined on the basis of geology and ground conditions, proposed structures, and anticipated construction methods. Alignment and proposed structures are current as of October 26, 2023. Geology and ground conditions were interpreted from available boring information and are shown in the Alternative F Tunnel Geologic Profile in Plate 10.

Reach locations are shown in Plate 10, and reach limit stationing, proposed structures, and general ground conditions are summarized in Table 18.

Reach limits were defined as follows:

- Limits of Reaches 1 and 5 were defined based on limits of proposed South Portal and North Portal approaches, respectively. Reach 1 includes an at-grade section with a fill wall section on the downslope slide (RW 1) and a cut section approach. Reach 5 includes an approach with an architectural arch and retaining walls (RW 3R and RW 3L), the Wilson Creek Tributary Bridge, and an at-grade section.
- The limit of Reach 2 was defined based on limits of the proposed South Portal cut-and-cover section, including retaining walls (RW 2R and RW 2L) and an EDAS.
- Limits of Reaches 3 and 4 were defined based on limits of the proposed tunnel to be constructed by SEM. In Reach 3 the tunnel will be excavated primarily in Melange of the Franciscan Complex, and in Reach 4 the tunnel will be excavated primarily in the Broken Formation of the Franciscan Complex.

Reach stationing shown in Table 18 applies to the tunnel or roadway centerline. The general ground descriptions for reach apply to the full width of the alignment.

Table 18. Alternative F Reach Descriptions

Reach [1]	Proposed Structures [2]	General Ground Conditions within Proposed Excavation	Approximate Stationing		Approximate Length Along Alignment (feet)	
			From	To		
1	Roadway at grade	• No excavation • Underlain by Earthflow	34+36	45+00	1,064	1,864
	South Portal Approach, Cut Section	• Earthflow	45+00	53+00	800	
2	South Portal Cut-and-Cover Section, with EDAS	• Earthflow underlain by Franciscan Complex Melange	53+00	58+00	500	500
3	SEM Tunnel	• Full-face excavation in Franciscan Complex Melange • Contact between Melange and Broken Formation at ~Station 67+20, oblique to alignment	58+00	67+20	920	920
4	SEM Tunnel	• Full face excavation in Franciscan Complex Broken Formation	67+20	116+73	4,953	4,953
5	North Portal Approach (Architectural Arch and Retaining Walls 3R and 3L)	• Thin (<5 ft) Colluvium underlain by Franciscan Complex Broken Formation	116+73	119+25	252	1,091
	Wilson Creek Tributary Bridge	• No excavation • Bridge foundations to be constructed through thin (<5 feet) Alluvium and Colluvium to Franciscan Complex Broken Formation	119+25	120+47	122	
	Roadway at grade	• No excavation • Underlain by thin (<5 feet) Fill/Colluvium and Franciscan Complex Broken Formation	120+47	127+64	717	
					Total length (feet)	9,328

Notes:
(1) Reaches were defined based on geology and ground conditions, proposed structures, and anticipated construction methods.
(2) Proposed structures, alignment, and stationing are current as of October 26, 2023.

The Tunnel Horizon for the proposed SEM tunnel was defined as a zone of tunnel excavation which extends 44.5 feet upward from the tunnel lower excavation limit for the design current as of October 26, 2023. Any overcut beyond the periphery of the permanent tunnel structure was considered outside of the Tunnel Horizon.

The Excavation Horizon for the cut and retained excavations and for the South Portal cut-and-cover section was defined as a zone of excavation which extends upward from the base of the invert slab to the ground surface. General ground conditions within the Tunnel Horizon and the Excavation Horizon are included in the reach descriptions in Table 18.

- *Distribution of Site Ground Classes*

The anticipated distribution of tunneling ground classes for Alternative F was estimated based on projections of available subsurface data, geologic mapping, and geologic interpretation. Table 19 shows the estimated distribution of Ground Classes within each reach for all proposed Alternative F excavation and also for just SEM tunnel excavation. The table also shows the estimated distribution of Ground Class Groups for all proposed Alternative F excavation and for just SEM tunnel excavation. All percentages shown are by volume.

Table 19. Estimated Ground Class Distribution for Alternative F

<table>
<tr><th colspan="8">Ground Classes</th><th colspan="3">Ground Class Groups</th></tr>
<tr><th rowspan="2">Ground Class</th><th colspan="5">Percent Volume for Reach</th><th rowspan="2">Percent Volume for All Excavation</th><th rowspan="2">Percent Volume for SEM Tunnel Excavation</th><th rowspan="2">Ground Class Group</th><th rowspan="2">Percent Volume for All Excavation</th><th rowspan="2">Percent Volume for SEM Tunnel Excavation</th></tr>
<tr><th>1</th><th>2</th><th>3</th><th>4</th><th>5</th></tr>
<tr><td>Fill, Alluvium, Colluvium (F, Qal, Qc)</td><td>0.0</td><td>0.0</td><td>0.0</td><td>0.0</td><td>46.2</td><td>0.7</td><td>0.0</td><td>Overburden</td><td>0.7</td><td>0.0</td></tr>
<tr><td>V (Qlsd-m)</td><td>100.0</td><td>85.7</td><td>0.0</td><td>0.0</td><td>0.0</td><td>13.5</td><td>0.0</td><td rowspan="2">Landslide Deposits</td><td rowspan="2">13.5</td><td rowspan="2">0.0</td></tr>
<tr><td>IV (Qlsd-bf)</td><td>0.0</td><td>0.0</td><td>0.0</td><td>0.0</td><td>0.0</td><td>0.0</td><td>0.0</td></tr>
<tr><td>III (KJFm)</td><td>0.0</td><td>14.0</td><td>100.0</td><td>0.0</td><td>0.0</td><td>15.0</td><td>15.7</td><td>Franciscan Complex, Melange</td><td>15.0</td><td>15.7</td></tr>
<tr><td>II (KJFbf-II)</td><td>0.0</td><td>0.0</td><td>0.0</td><td>50.0</td><td>53.4</td><td>35.8</td><td>42.2</td><td rowspan="2">Franciscan Complex, Broken Formation</td><td rowspan="2">70.7</td><td rowspan="2">84.3</td></tr>
<tr><td>I (KJFbf-I)</td><td>0.0</td><td>0.0</td><td>0.0</td><td>50.0</td><td>0.0</td><td>34.9</td><td>42.2</td></tr>
</table>

Notes:
1. Alignment and geologic profile are current as of October 26, 2023.
2. Assume ground conditions are uniform across the full width of the alignment including portal approaches, cut-and-cover section, and SEM tunnel.
3. Assume overburden Ground Classes Fill, Alluvium, and Colluvium are locally intermixed.
4. Assume equal proportions of Ground Classes I and II in Reach 4.
5. Assume no Ground Class I in Reach 5.

The Alternative F geologic profile in Plate 10 shows the spatial distribution of Ground Class Groups along the length of the Alternative F alignment. Because this is an interpreted profile, with information extrapolated and interpolated over between widely spaced borings, actual ground conditions will likely differ from the conditions shown.

Although based on the limited available subsurface data, the distributions and relative proportions of Ground Classes and Ground Class Groups shown in Table 19 and discussed in the following sections are suggested for initial planning purposes. They will be refined as additional data become available.

- *Site Tunneling Conditions by Reach*

As discussed, five reaches were defined for the Alternative F alignment. Reach limits were defined based on ground conditions, proposed structures, and anticipated construction method. Reach locations are shown in profile in Plate 10 and are described in Table 18. Groundwater conditions are discussed in Section 3.7.4.

All stationing and structure locations are from the design alignment and configuration current as of October 26, 2023. Preliminary geotechnical ground characterization by reach is discussed in the following sections.

<u>Reach 1 (South Portal Approach)</u>

As shown in Plate 10, the southern portion of Reach 1 would be roadway at grade as it diverges from existing U.S. 101. The proposed structure to be constructed in Reach 1 would consist of an 800-foot length of cut slope from Station 45+00 to 53+00. A fill wall (RW 1) would be constructed on the downhill side of the roadway.

Excavation in Reach 1 is anticipated only in the northern 100 feet of the reach, where the maximum depth of excavation anticipated to be about 33 feet. Elsewhere in the reach the roadway would be at or slightly below existing grade.

As shown in Plate 10, all material to be excavated in Reach 1 is anticipated to be Ground Class V Earthflow (Qlsd-m).

Groundwater depths in Reach 1 are expected to range from approximately 25 feet BGS at the south end to 60 to over 70 feet BGS at the north end, based on the limited available information. However, it is important to note that depths to groundwater calculated based on VWP pressure-related groundwater elevations may not represent the actual depth of groundwater in the subsurface; additional exploration is required to evaluate actual groundwater depths along the alignment. High (>500 gallons per minute per 1,000 feet) groundwater inflows are not anticipated in the surface-based excavations in Reach 1. Groundwater inflow estimates will be re-evaluated after additional explorations have been completed.

Geotechnical properties of the earthflow deposits are discussed in Sections 3.4 and 3.5. Groundwater conditions for South Portal area are discussed in Section 3.7.4.3.

<u>Reach 2 (South Portal Cut-and-Cover Section)</u>

As shown in Plate 10, the proposed structure to be constructed in Reach 2 consists of the 500-foot-long South Portal cut-and-cover approach section from Station 53+00 to 58+00, including the EDAS and two retaining walls (RW 2R and RW 2L). Depth of excavation in Reach 2 would range from about 33 feet at the south end of the reach to about 149 feet at the north end.

As shown on Plate 10 and in the ground class distribution summary in Table 9, the material to be excavated in Reach 2 would consist mostly of Ground Class V earthflow (86 percent), underlain by Ground Class III Franciscan Complex Melange below the basal failure surface of the earthflow. Excavation of the Melange would be within the northern 220 feet of the reach, where its maximum excavated thickness would be about 60 feet, constituting about 14 percent of excavated volume for Reach 2.

The proposed secant piles for the cut-and-cover section would be socketed into the Mélange well below the earthflow basal failure surface. The collapsible columns of the EDAS would also extend through the zone of earthflow movement into the underlying Melange.

Groundwater depths in Reach 2 are expected to range from approximately 60 to over 70 feet BGS, based on the limited available information. High groundwater inflows are not anticipated in excavations in Reach 2 because weathering of the argillite of the Melange is likely to have effectively reduced fracture flow, but open fractures producing significant inflows are possible. Such flows could be sustained and under pressure. Additional explorations would be necessary to determine if this condition could be present in the Melange and specifically within Reach 2.

Geotechnical properties of the earthflow deposits and the Ground Class III Melange are discussed in Sections 3.4 and 3.5. Groundwater conditions for South Portal area are discussed in Section 3.7.4.3.

<u>Reach 3 (SEM Tunnel)</u>

As shown in Plate 10, the proposed structure to be constructed in Reach 3 consists of a 920-foot length of SEM tunnel, from Station 58+00 to 67+20. The planned spring line width of the tunnel liner is 66.25 feet, based on the design current as of October 26, 2023. Crown depth in Reach 3 ranges from about 149 feet BGS near the south limit of the reach at Station 58+00 to about 229 feet BGS at the north limit of the reach at Station 67+20.

The boundary between Reach 3 and Reach 4 was defined on the basis of geology as interpreted from currently available data. A steeply dipping contact between the Franciscan Complex Melange and the Broken Formation is inferred in the vicinity of Station 67+20. Orientation of the contact is not known, nor is its nature as a boundary, which could be abrupt or several hundred feet wide and gradational or sheared.

As shown on Plate 10 and in the ground class distribution summary in Table 19, SEM excavation in Reach 3 would be entirely in Ground Class III Franciscan Complex Melange, in full face excavation.

Groundwater depths in Reach 3 are expected to range from approximately 60 to over 70 feet BGS at the south end to possibly deeper toward the north end, based on the limited available information; there are no exploration locations along the alignment north of the portal area. High

(>500 gallons per minute per 1,000 feet) groundwater inflows are not anticipated in the surface-based excavations in Reach 3. Groundwater inflow estimates will be re-evaluated after additional explorations have been completed.

Geotechnical properties of the Ground Class III Melange are discussed in Sections 3.4 and 3.5. Groundwater conditions for the SEM tunnel are discussed in Section 3.7.4.2.

Reach 4 (SEM Tunnel)

As shown in Plate 10, the proposed structure to be constructed in Reach 4 consists of a 4,953-foot length of SEM tunnel, from Station 67+20 to 116+73. The planned outside diameter of the tunnel liner is the same as for Reach 3 at 44.5 feet. Based on the design current as of October 26, 2023, crown depth in Reach 4 ranges from about 229 feet BGS at Station 67+20 at the south limit of the reach to about 53 feet BGS at Station 101+80 in the north central part of the reach.

As shown on Plate 10 and in the ground class distribution summary in Table 19, excavation in Reach 4 is anticipated to be entirely in the Ground Class I and II Franciscan Complex Broken Formation. The geologic unit was subdivided into two ground classes, based on inferred block size, weathering, and strength. Ground Class I has larger blocks with higher intact rock strength and less weathering than Ground Class II. Because the distribution of these ground classes could not be determined from currently available information, Reach 4 was assumed to have equal portions of Ground Class I and Ground Class II within the Tunnel Horizon. The two ground classes may be randomly mixed along the length of the reach.

The limited available data for Reach 4 indicate that calculated groundwater levels are approximately 150 feet BGS and deeper for much of the reach but may be as shallow as 10 to 20 feet BGS at the north end. Most SEM excavation in Reach 4 may be dry, except for infiltrated surface runoff and surface water collected in the northeast-draining drainage way which crosses the alignment at the north limit of SEM construction. This suggests that high hydrostatic pressures are not likely to act on the tunnel lining. Additional data are needed to better characterize groundwater conditions in Reach 4.

Observations of water loss during drilling indicate that open fractures in sandstone producing large volumes of groundwater could be present in Reach 4. The degree of fracture connectivity is not currently known, but high (>500 gallons per minute per 1,000 feet) flush flows should be anticipated. Additional site exploration and testing would help to confirm if high-flow fracture zones are present.

Geotechnical properties of Ground Class I and II Broken Formation are discussed in Sections 3.4 and 3.5. Groundwater conditions for the SEM tunnel are discussed in Section 3.7.4.2.

Reach 5 (North Portal Approach)

As shown in Plate 10, the proposed structures to be constructed in Reach 5 would be a portal structure with architectural arch and two retaining walls (RW 3R and RW 3L) and the Wilson Creek Tributary Bridge.

Excavation in Reach 5 is anticipated only for the portal approach structure in the southern 252 feet of the reach, where the maximum depth of excavation anticipated to be about 75 feet with additional scaling of loose Colluvium or rock. Elsewhere in the reach the roadway would be at or slightly below existing grade.

Depth of excavation in Reach 5 ranges from about 35 feet at the headwall at the south end of the reach at Station 116+73 to near zero at the south end of the bridge. Maximum height of the roadway above existing ground level beneath the planned bridge is about 28 feet.

As shown in Plate 10, excavated material in Reach 5 is anticipated to be about 46 percent Colluvium Ground Class V Landslide Deposits derived from the Broken Formation and 54 percent Ground Class II Franciscan Complex Broken Formation. Soldier piles installed to retain excavations in Reach 5 would be anchored in Ground Class II Broken Formation. Bridge foundations would bear on Ground Class II Broken Formation.

Based on the limited available data, groundwater levels in Reach 5 are expected to be 10 to 20 feet BGS but locally higher. High (>500 gallons per minute per 1,000 feet) groundwater inflows are not anticipated in excavations in Reach 5.

Geotechnical properties of Ground Class V Landslide Deposits and Broken Formation are discussed in Sections 3.4 and 3.5. Groundwater conditions for the North Portal area are discussed in Section 3.7.4.3.

3.7.4 Site Groundwater Conditions – Alternative F

No borings have been drilled to date along the Alternative F alignment and no VWPs installed to monitor alignment-specific groundwater conditions except near the portals. Nine VWPs were selected as representative of groundwater conditions for Alternative F (Table 20), based on their proximity to the alignment (projected perpendicularly), transducer(s) near the proposed tunnel alignment elevation, and projected geologic and hydrogeologic conditions.

Table 20. Alternative F Groundwater Information

Boring ID	Total Bore Depth (feet)	Surveyed Ground Surface Elevation (feet)	Approximate Projected Alternative F Location	Packer Testing		Transducer Depth (feet)	Transducer Elevation (feet)	Apparent Groundwater Depth Minimum (feet)	Apparent Groundwater Elevation Maximum (feet)
				K value (ft/sec)	Test Interval Depth (feet)				
P-19-007	305	585.5	South Portal Area	--	--	295	290.5	218.5	367.0
						195	390.5	147.8	437.7
						95	490.5	82.6	502.9
RC-20-006	251	619.3	South Portal Area	--	--	199.5	419.8	73.5	545.8
						129	490.3	65.3	554.0
						60	559.3	58.2	561.1
RC-20-005	250	859.1	SEM Tunnel	--	--	250	609.1	216.6	642.5
						232	627.1	205.6	653.5
						155	704.1	142.9	716.2
RC-20-014	300	805.1	SEM Tunnel	6.22E-08	290 to 300	290	515.1	167.0	638.1
				4.19E-07	220 to 230	225	580.1	167.0	638.1
				Failed Test	163 to 173	166	639.1	147.8	657.3
RC-20-015	301	883.4	SEM Tunnel	--	--	290	593.4	241.0	642.4
						255	628.4	149.4	734.0
						159	724.4	146.7	736.7
RC-20-017	300	829.4	North Portal Area	Failed Test	275 to 285	282	547.4	225.9	603.5
						253	576.4	221.8	607.6
				1.88E-06	206 to 216	217	612.4	207.5	621.9
				4.57E-07	170 to 180	182	647.4	177.8	651.6
						150	679.4	137.9	691.5
RC-19-003	100	840.5	North Portal Area	--	--	90	750.5	11.6	828.9
RC-20-013	135	830.5	North Portal Area	--	--	133	697.5	82.5	748.0

3.7.4.1 South Portal Area Groundwater Conditions

The VWPs selected to evaluate groundwater conditions at the proposed South Portal area and approach were RC-20-019, P-19-007, and RC-20-006. The transducers in RC-20-006 are located above and below the landslide failure surface. The apparent groundwater elevation is expected to be approximately 450 feet near Station 44+50 and approximately 561 feet near the portal. VWP RC-20-006 (Figure 16) shows typical signatures from transducers located below and above the EF landslide mass.

As discussed in Section 3.3, groundwater measurements in the VWPs in this area are variable and show a downward hydraulic gradient in each case, indicative of discharge toward the ocean.

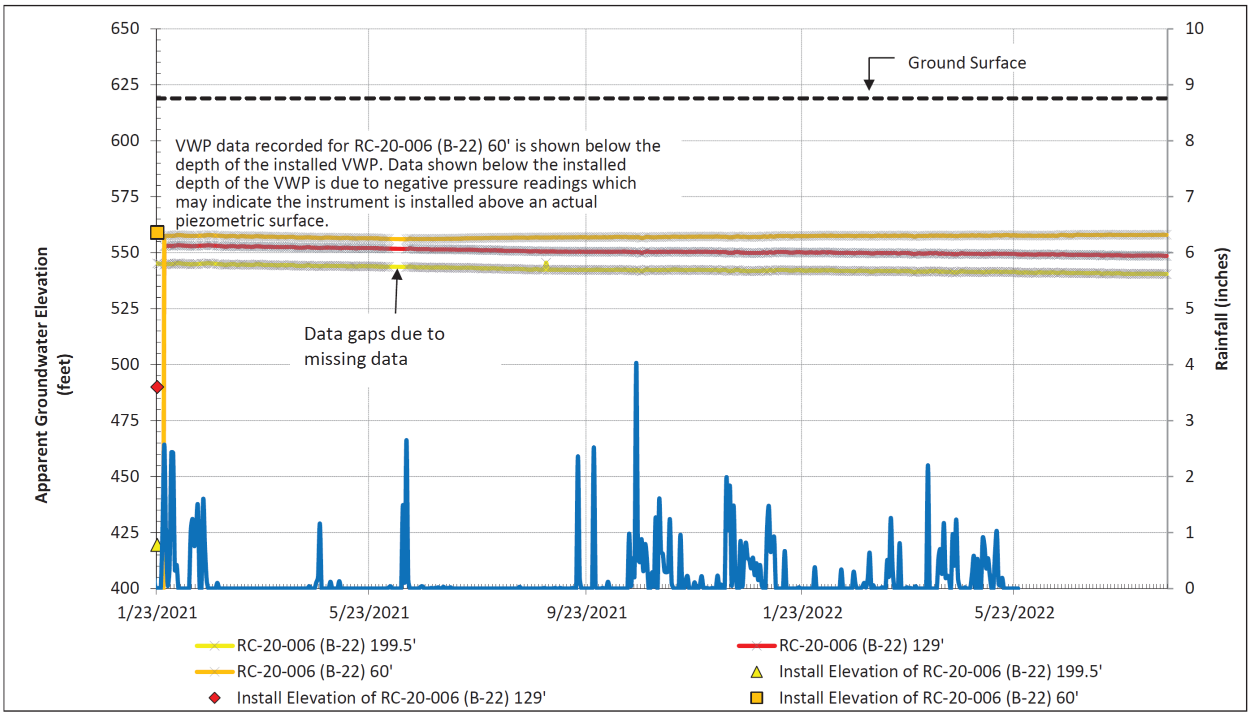


Figure 16. VWP RC-20-006 Hydrograph Showing Multiple Transducers and Rainfall

3.7.4.2 SEM Tunnel Groundwater Conditions

For the portion of the tunnel alignment within the Melange, the groundwater is probably dominated by fracture flow. The data for VWPs at RC-20-014 indicate similar head values for the two deeper VWPs and a small upward gradient at these depths (Figure 17). For the portion of the tunnel alignment within the Broken Formation, VWPs recorded apparent groundwater elevations above and below the alignment.

No alignment-specific test data are yet available to evaluate hydraulic properties of bedrock at tunnel depth. Hydraulic conductivity estimated from packer tests in applicable borings is 4.19×10^{-7} and 6.22×10^{-8} feet/second for the test intervals at depths of 220 to 230 feet and 290 to 300 feet, respectively (Table 20). Hydraulic conductivity may be locally higher or lower than indicated by packer test results, and fracture intervals are likely to have the highest conductivity.

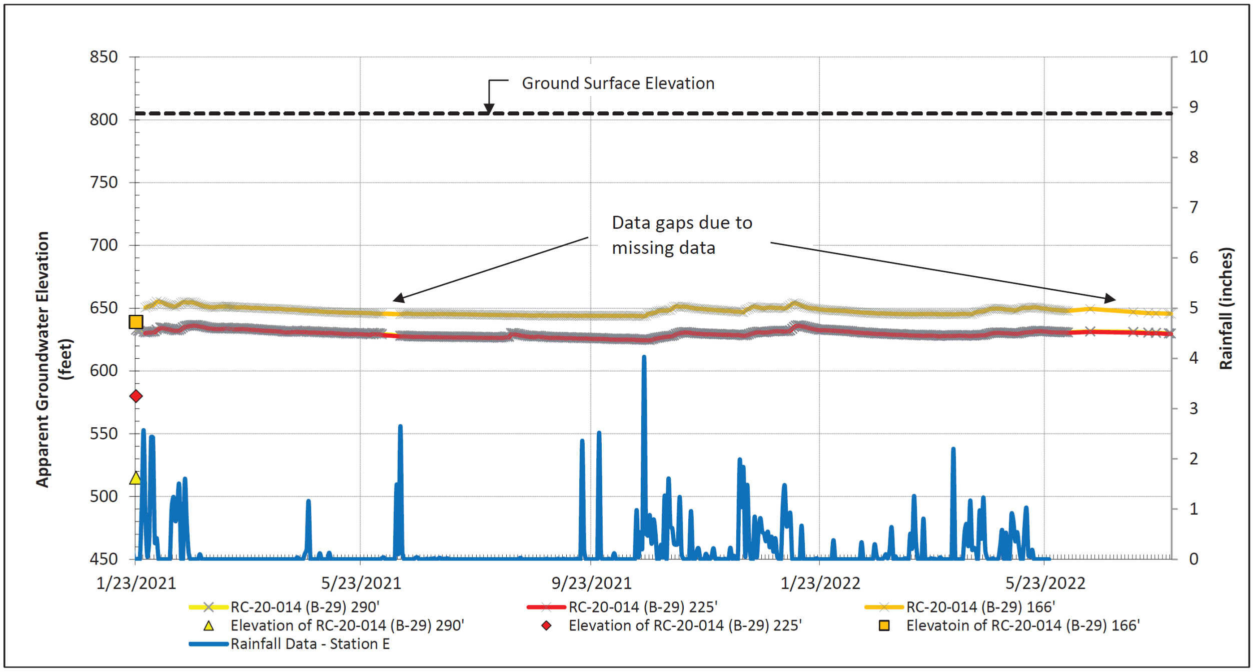


Figure 17. VWP RC-20-014 Hydrograph Showing Multiple Transducers and Rainfall

3.7.4.3 North Portal Area Groundwater Conditions

Data from VWPs RC-19-003, RC20013, and RC-20-017 were considered to characterize groundwater conditions in the North Portal area (Table 20). VWP RC-20-017 is located at an elevation of 829.4 feet and has maximum apparent groundwater elevation of 691.5 feet (Figure 18). VWP RC-19-003 has the shallowest apparent groundwater level (elevation of 828.4 feet). This apparent groundwater elevation is below the Alternative F alignment elevation by approximately 17 feet. However, the original ground surface above the Alternative F alignment elevation (before excavation) is approximately 160 feet higher than the VWP RC-19-003 ground surface. It is possible that the groundwater head could be higher at the alignment than the data projected from the VWP RC-19-003 and could be above the lower excavation limit in the North Portal area.

No alignment-specific test data are yet available to evaluate hydraulic properties of materials in the North Portal area. Hydraulic conductivity estimated from packer tests in applicable borings is 4.07×10^{-7} and 1.88×10^{-6} feet/second for the test intervals at depth 220 to 230 feet and 290 to 300 feet, respectively (Table 20). Hydraulic conductivity may be locally higher or lower than indicated by packer test results, and fracture intervals are likely to have the highest conductivity.

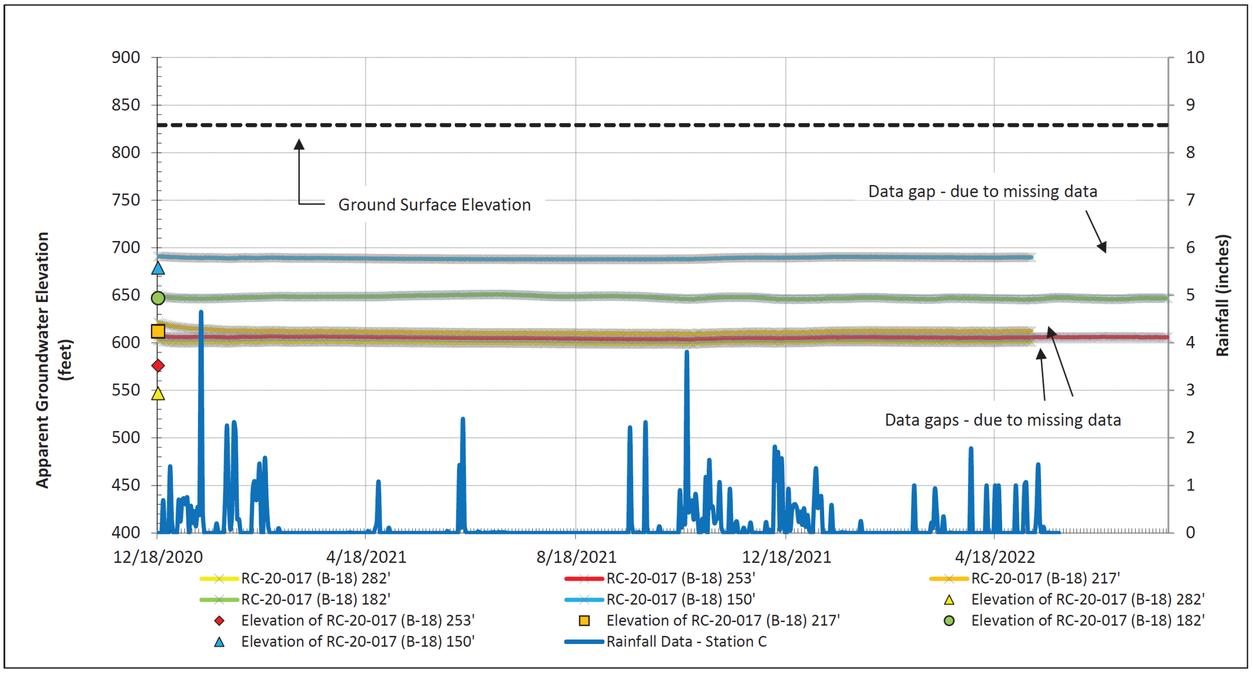


Figure 18. VWP RC-20-017 Hydrograph with Rainfall

3.7.5 Preliminary Seismic Hazard Evaluation – Alternative F

Preliminary seismic hazard evaluation for Alternative F is described in this section. It should be noted that all recommendations provided in this section shall be further confirmed or revised when site-specific data from future field investigations become available and more detailed analyses are performed.

3.7.5.1 Site Seismic Parameters

Site seismic parameters were evaluated for the structures along the alignment. According to the current plans and drawings, in addition to the tunnel, two retaining walls and one bridge are proposed along the Alternative F alignment. For the tunnel, the two portals and the middle location were used for seismic hazard evaluation. As a result, four sites were used for seismic evaluations along the Alternative F alignment.

- *Sites Where Shear-Wave Velocity V_{S30} Values are Evaluated*

Table 21 lists the geospatial coordinates (latitude/longitude) of the four selected sites along the Alternative F alignment, where V_{S30} was evaluated. The geospatial site coordinates were also estimated according to the Google Maps using the current project plans.

Table 21. Preliminary Site Seismic Parameters (Alternative F)

Structure(s)	RW 1R/1L	Tunnel/ South Portal/ RW 2R/2L	Tunnel Middle	Tunnel/ North Portal RW 3R/3L Bridge
Station Range (1)	"F" 45+00 to "F" 49+00	"F" 52+00.00 to "F" 58+00	"F" 85+00	"F" 116+72.69 to "F" 120+00
Reference Boring(s) (2)	RC-19-001 RC-20-006		-	RC-19-003 RC-20-013 RC-20-017
Site Geospatial Coordinates (latitude, longitude) (3)	41.624°, -124.1115°	41.624°, -124.1115°	41.6344°, -124.1105°	41.6425°, -124.1146°
V_{S30} (m/s)	280	280	1,149 (4)	340

Notes:
(1) Based on *20220701_GAD_Draft.pdf* plan sheet.
(2) Based on *2022-0708_SUB032_Prelim-Geotech-Data-Report_Final-2.pdf* (Caltrans, 2022a).
(3) Estimated per Google Maps and the current Geometric Approval drawings.
(4) Estimated from the nearby P- and S-wave suspension logging data.

- *Time-Averaged Shear-Wave Velocity V_{S30} for Top 30 m of Earth Materials*

Shear wave velocity for the alignment was measured at limited locations using direct measurement in boreholes (P- and S-wave suspension logging), and geophysical surveys (seismic refraction survey). Due to limited number of these measurements and their distance from the structure sites, the V_{S30} values were also estimated in accordance with the Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations (Caltrans, 2012 and 2021b), based on the correlation with SPT blow counts recorded in the nearby borings. The estimated V_{S30} values from different explorations for Alternative F are listed in Table 22. Details are provided in Appendix B-1.

After comparison with the existing limited P- and S-wave suspension logging and seismic refraction measurements, it was observed that the 2021 Caltrans correlations tend to yield a much lower value of V_{S30} than shear wave velocity estimates from seismic refraction lines. The 2012 correlations result in V_{s30} estimates that better match the values form seismic refraction lines. Therefore, for preliminary evaluations, the V_{S30} values estimated from SPT blow counts using 2012 Caltrans procedures have been used for structure sites. The V_{s30} values will be verified once site-specific geotechnical investigation is performed for each structure.

Table 22. Estimated V_{S30} from Geotechnical/Geophysical Explorations (Alternative F)

Type of Geotechnical Exploration	Exploration ID	Estimated V_{S30} (m/sec)				
		From Field Blow Counts		P- and S-Wave Suspension Logging		From Seismic Refraction
		Caltrans 2012 Method (1)	Caltrans 2021 Method (2)	R1-R2 (3)	S-R1 (4)	
Soil Boring	RC-18-001	308	248	NA	NA	NA
	RC-18-007	302	270	NA	NA	NA
	RC-18-009	288	228	NA	NA	NA
	RC-19-001	284	NA	NA	NA	NA
	RC-19-003	397	NA	NA	NA	NA
	RC-20-006	270	202	NA	NA	NA
	RC-20-013	309	264	NA	NA	NA
	RC-20-017	361	324	NA	NA	NA
Seismic Refraction Line	SL-11	NA	NA	NA	NA	594
	SL-12	NA	NA	NA	NA	565
	SL-13A	NA	NA	NA	NA	427
	SL-13B	NA	NA	NA	NA	338
	SL-42	NA	NA	NA	NA	456
P- and S-Wave Suspension Logging	R-20-014 (Calculated from 200 feet to 287 feet BGS)	NA	NA	2,827	2,624	NA
	R-21-011 (Calculated from 210 feet to 293.5 feet BGS)	NA	NA	1,972	2,031	NA
	R-20-019 (Calculated from 0 feet to 80.3 feet BGS)	NA	NA	546	598	NA

<u>Notes:</u>

(1) V_{S30} calculated per Caltrans (2012).
(2) V_{S30} calculated as per Caltrans (2021).
(3) Estimated from the nearby P- and S-wave suspension logging data using R1-R2 Methodology.
(4) Estimated from the nearby P- and S-wave suspension logging data using S-R1 Methodology.

3.7.5.2 Ground Motion Parameters

Ground motion parameters were evaluated in general accordance with Caltrans Geotechnical Manual – Design Acceleration Response Spectrum module (Caltrans, 2020a). As discussed in Section 3.6.5.2, Caltrans current practice is to use the SEE design ARS developed per Caltrans SDC 2.0 (2019a) to characterize design ground motions for ERSs, embankments, slopes, sign structures and other appurtenant highway facilities.

Based on the procedures described in Caltrans SDC 2.0 (2019a) and October 2019 Interim Revisions to SDC 2.0 (2019b), the preliminary ARS curves for SEE with a 975-year return period at the four selected sites were determined using the Caltrans ARS Online V3.0.2 (2021c), utilizing the site coordinates and V_{S30} values shown in Table 21 and Table 22. The resulting preliminary ground motions parameters and ARS are provided in Table 23. Details are provided in Appendix B-2.

- *Horizontal Peak Ground Acceleration (HPGA)*

The preliminary HPGA values for the four selected sites are summarized in Table 23. For the above-ground structures and tunnel portals, the HPGA values are nearly identical, varying from 0.85g to 0.87g. For the mid-tunnel site, due to higher V_{S30} value, the HPGA is 0.65g.

Table 23. Preliminary Ground Motion Parameters (Alternative F)

	Structure(s)	RW 1	Tunnel/ South Portal-RW 2R/2L	Tunnel/Middle	Tunnel/ North Portal-RW 3R/3L
Site Geospatial Coordinates (latitude/longitude)		41.624°, -124.1115°	41.6262°, -124.1109°	41.6344°, -124.1105°	41.6425°, -124.1146°
Horizontal Spectral Acceleration (g)	PGA	0.880	0.870	0.650	0.850
	0.1 sec	1.400	1.400	1.390	1.430
	0.2 sec	1.730	1.730	1.320	1.760
	0.3 sec	1.890	1.890	1.090	1.820
	0.5 sec	1.760	1.760	0.770	1.570
	0.75 sec	1.510	1.510	0.600	1.310
	1 sec	1.250	1.240	0.500	1.100
	2 sec	0.660	0.660	0.260	0.560
	3 sec	0.400	0.400	0.160	0.340
	4 sec	0.270	0.270	0.110	0.230
	5 sec	0.180	0.180	0.080	0.160
Mean Earthquake Moment Magnitude		8.65	8.65	8.58	8.67
Mean Site to Fault Source Distance for S_a at 1 second (km)		20.0	20.0	20.3	20.1
Site Class		S1	S1	S1	S1
Horizontal Seismic Coefficient, k_h		0.29	0.29	N/A	0.28

- *Mean Earthquake Moment Magnitude (M)*

The preliminary M values for the four selected sites are also presented in Table 23. The M values along the Alternative F alignment are quite similar, varying from M8.58 to M8.67.

- *Mean Site to Fault Source Distance*

The preliminary Mean Site to Fault Source Distance values for the four selected sites are also presented in Table 23. The values along the Alternative F alignment are quite similar, varying from 20.0 to 20.3 kilometers.

3.7.5.3 Parameters for Seismic Slope Stability Analysis for Portal Area Slopes

According to the Caltrans Geotechnical Manual – Landslides (Caltrans, 2020b) and Caltrans Geotechnical Manual – Embankments (Caltrans, 2014), a horizontal seismic coefficient for seismic slope stability analysis equal to one-third of the horizontal peak horizontal acceleration at the site can be used for preliminary seismic slope stability evaluations. These horizontal seismic coefficients for the portal area slopes and the other surface structure sites are tabulated in Table 23. Alternatively, a displacement-based seismic slope stability approach similar to the one recommended in Caltrans Geotechnical Manual – Liquefaction-Induced Lateral Spreading module (Caltrans, 2020c) may be used to evaluate the seismic stability of landslides.

3.7.5.4 Parameters for Racking and Ovaling Analysis of Tunnel

During a seismic event, the tunnel is expected to remain in intimate contact with the surrounding ground and will be subjected to seismically induced deformation: racking for a rectangular tunnel or ovaling for a circular tunnel. Free-field shear strains of the surrounding ground will cause deformation of the tunnel section during an earthquake. To assist in the integrity analysis of the proposed tunnel section, earthquake-induced shear strains of the free-field ground shall be estimated. These shear strains can be determined by a comprehensive seismic response analysis or empirical correlations. For preliminary evaluations in this report, the latter method was adopted. The maximum shear strain (γ_{max}) of the free-field near the tunnel horizon can be estimated as the ratio of the maximum free-field velocity (V_{max}) to the average shear wave velocity (V_S) of the surrounding ground. Integrating γ_{max} from the bottom of the tunnel to the ground surface will yield the corresponding seismically induced lateral displacement profile. This displacement profile can be then imposed on the boundary of a numerical analysis model (e.g., a finite element model) to evaluate the mechanical response of the subject tunnel-lining system.

The preliminary V_S value of the surrounding ground for the tunnel section can be evaluated from the existing P- and S-wave suspension logging data in nearby borings. The V_{max} value for the same location can be estimated using the on-site ground motion parameters and the correlations recommended in the National Cooperative Highway Research Program (NCHRP) Report 611 (Anderson et al., 2008), or other similar correlations. For preliminary evaluations, a V_{max} of 0.8 meters per second (m/s), based on the correlations from NCHRP Report 611, can be used. It should be noted that due to proximity of the site to CSZ, ground motions at the site could have different characteristics than seismic regions that are controlled by shallow crustal earthquakes. Therefore, any correlation developed based on shallow crustal earthquake records shall be used with care and for preliminary evaluations only.

For advanced design of the tunnel, more detailed analyses using a finite elements or site response model, using site-specific strong motion records are recommended. By using site-specific strong motion records, the project seismic settings, e.g., magnitude, distance and seismic source types can be incorporated in the design.

3.7.5.5 Fault Rupture

Fault rupture hazard was evaluated in accordance with Caltrans Geotechnical Manual – Fault Rupture module (Caltrans, 2017) and Caltrans MTD 20-10 (Caltrans, 2013). MTD 20-10 requires a SFRDHA where any portion of the structure is located:

- Within an APEFZ, as defined by the CGS.
- Within 1,000 feet of an unzoned fault (not located in an APEFZ) that is Holocene (11,000 years) or younger in age.

The alignment of Alternative F does not transverse within 1,000 feet of any active faults as delineated by the APEFZ (CGS, 2007) or UCERF3 model. Therefore, based on the current Caltrans criteria, the potential for surface ground rupture along the subject alternative alignment is negligible.

3.7.5.6 Liquefaction

Liquefaction hazard was evaluated in accordance with Caltrans Geotechnical Manual – Liquefaction module (Caltrans, 2020b) and using SPT blow counts.

Preliminary liquefaction potential analysis was performed using the procedures outlined by Youd and Idriss (2001), the blow counts and measured groundwater depths of the existing nearby borings, and the preliminary ground motion parameters listed in Table 23. The seismically induced settlements were also estimated using the empirical method proposed by Tokimatsu and Seed (1987). The preliminary results are summarized in Table 24. Due to the presence of deep groundwater or fine-grained soils above groundwater, no liquefiable layers are identified. The larger seismically induced settlements at some locations are also mainly derived from the dry sand settlements above groundwater and at shallow depths of approximately 3 to 15 feet. If the proposed footing bottom elevation of a retaining wall is placed deeper than these depths, seismically induced settlements affecting the structure will be lower.

Table 24. Summary of Preliminary Liquefaction Analysis Results (Alternative F)

Structure(s)	Reference Boring(s)	Surveyed Ground Surface Elevation (feet)	Measured Groundwater Elevation (1) (feet)	Moment Magnitude	HPGA (g)	Liquefiable Layer Elevations (feet)	Seismically Induced Settlement (inch)
Tunnel / South Portal / RW 2R/2L	RC-19-001	+538.8	+478.8	8.65	0.88	None	0.7
	RC-20-006	+619.3	+559.3	8.65	0.87	None	11.1 (2)
Tunnel / North Portal / RW 3R/3L / Bridge	RC-19-003	+840.5	+830.5	8.67	0.85	None	<0.1
	RC-20-013	+830.5	+697.5	8.67	0.85	None	0.4 (2)
	RC-20-017	+829.4	+679.4	8.67	0.85	None	1.7 (2)

Notes:
(1) Groundwater elevation (shallowest measured) extracted from Final Preliminary Geotechnical Data Report (Caltrans, 2022a).
(2) Mainly dry-sand settlement.

3.7.5.7 Liquefaction-Induced Lateral Spreading and Other Lateral Spreading Conditions

Liquefaction-induced lateral spreading hazard shall be evaluated in accordance with Caltrans Geotechnical Manual – Liquefaction-Induced Lateral Spreading module (Caltrans, 2020c). Based on the current information, the potential for lateral spreading at structure sites along the Alternative F alignment is low, as these locations are not underlain by soil susceptible to liquefaction.

4 GEOTECHNICAL DESIGN EVALUATION

4.1 Alternative X

4.1.1 Evaluation of Alternative X Design Components

4.1.1.1 Retaining Wall Structures

Anchored soldier pile walls with lagging panels are recommended for four new walls, one in a fill section and three in cut sections. The anchored soldier pile ERS at cut slopes would be in a stacked configuration with terraces (benches) at the top of lower tiered walls, as presented on Plate 1b. ERS type selection considered the practicality and feasibility/constructability for the topographic and geologic conditions along Alternative X, and past performance of similar ERS types in similar conditions and movement (settlement and deformation) tolerances of various ERSs. Based on this assessment, ERS types such as concrete gravity wall, rock gravity wall, gabion basket wall, soil nail wall, sheet pile wall, soil-cement mixed wall, and slurry diaphragm walls were deemed unsuitable.

4.1.1.2 Anchored Soldier Pile Walls with Concrete Lagging Panels and Ground Anchors

- *Description*

For conceptual design, the anchored soldier pile retaining walls (RW 6 fill wall and RW 7A through RW 7C cut walls, as summarized in Table 1) are anticipated to consist of steel soldier piles installed at 8- to 10-foot spacing and embedded in concrete shafts at least several feet below final excavation grade to provide passive toe resistance for wall support. Where walls are present on both sides of the highway, as shown in Figure 19, the lower tiered, cut walls would be spaced at least 42 feet behind the fill wall to accommodate travel lanes and shoulders.

Where stacked (terraced) walls are planned, as shown in Figure 20, the upper tiered, cut walls would be spaced 60 feet set back behind the lower tiered, cut walls. Timber lagging panels would be placed between the steel pile flanges to retain the earth behind the piles. Permanent ground anchors would be grouted into drilled holes in the native ground. These types of ERSs are expected to be constructed in a top-down method by first installing the soldier piles in drilled holes filled with concrete, then installing the ground anchors and lagging as the excavation in front of the wall or backfill behind the wall progresses.

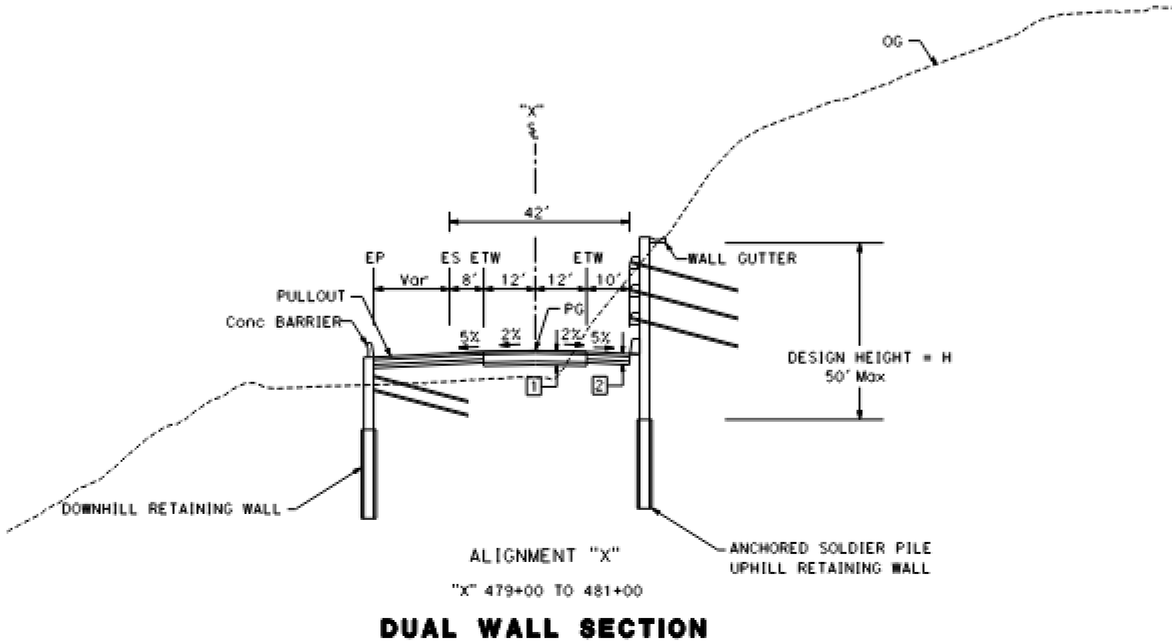


Figure 19. Typical Dual Wall Section for Alternative X

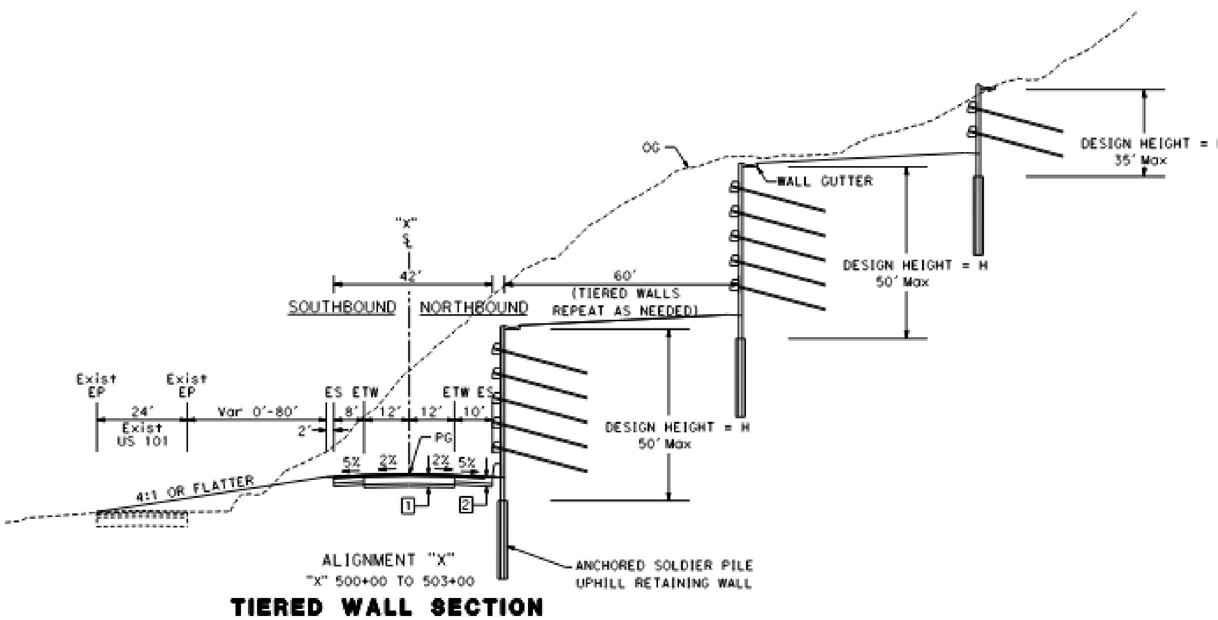


Figure 20. Typical Tiered Wall Section for Alternative X

- *Analysis and Results*

With consideration given to performance expectations, cost, site conditions, and constructability, anchored soldier pile walls are considered to be the most suitable and practical selection for the needed retaining walls. The ERSs using anchored soldier pile walls are expected to be founded on or in geologic units that will likely range from colluvium (Qc) to landslide deposits (Qlsd-m and Qlsd-bf). Anchored soldier pile ERSs are typically considered where substantial wall deformation/movements are anticipated.

- *Evaluation*

All anchored soldier pile wall features to be advanced and engineered during future design phases are expected to require special designs due to the stacked configuration and high levels of horizontal seismic ground motion (i.e., ½ PGA). Another consideration for design and assessment of adequacy of the anchored soldier pile ERSs on a wall-by-wall basis will include potential added loads from the large landslides on which they are founded. These non-gravity ERSs rely heavily on structural components of the vertical pile elements partially embedded in competent foundation material to mobilize resistance against lateral loads. The method of soldier pile installation is expected to be by cast-in-drilled-hole (CIDH) shafts type foundation construction.

Recommendations for ground anchor length and minimum soldier pile embedment depth will be based on geotechnical capacity requirements, and considerations such as socketing into competent material as well as meeting global stability requirements. Engineering evaluations during future design phases will also need to address lagging recommendations for timber plank embedment below finished grade, lagging dimensions, and section sizing. Geotechnical reports during the production design phase will also describe special provision type issues such as presence of groundwater, potentially difficult drilling and excavation conditions, and potential for caving.

4.1.1.3 Earthwork

- *Description*

Grading operations for earthwork along Alternative X will require excavation activities. Based on preliminary estimates for mass grading quantities by the Civil Engineers designing the roadways, earthwork for Alternative X will generate approximately 250,000 cubic yards (CY) of excess material. It is assumed that these materials will be hauled off-site to be disposed of at or near the Crescent City area. The design team has assumed that based on a haul truck capacity of 12 CY per load and a round-trip travel time of 1.5 hours, an estimated 10 trucks working over 440 working days (about 2 years) will be needed to remove this material.

- *Analysis and Results*

Based on interpretation of available data, excavation of the soil-like and weak rock portions of the colluvium and landslide deposit materials anticipated along Alternative X could be performed using conventional earthmoving equipment such as excavators and tracked bulldozers with rippers. Rock cut excavations, if any, would likely be performed by hoe-ram through the weathered and more heavily fractured rock; however, controlled rock blasting may be necessary if localized harder more resistant rock zones are encountered within the excavation limits.

The earthwork that would be necessary to construct the various retaining walls are not expected to improve overall stability (static and pseudo-static cases) of the deep-seated landslides along Alternative X.

- *Evaluation*

Detailed engineering analyses will be performed in subsequent design phases in order to advance earthwork design along Alternative X, and development of project-specific non-standard special provisions if deemed necessary. Design refinements would include items such as shear keys for fill embankments to be constructed on sloping ground, subdrainage, low height berms and v-ditches, and modifications to slope layback configurations and heights.

4.1.1.4 Underground Drainage System for Landslide Mitigation

- *Description*

As discussed in Section 1.5.3.3, the underground drainage system would include three drainage gallery tunnels constructed below the basal failure surface, with radial gravity drains drilled upward into the slide mass, three interconnected vertical shafts, and an outfall structure. The layout of the Alternative X underground drainage system is shown in Figure 21.

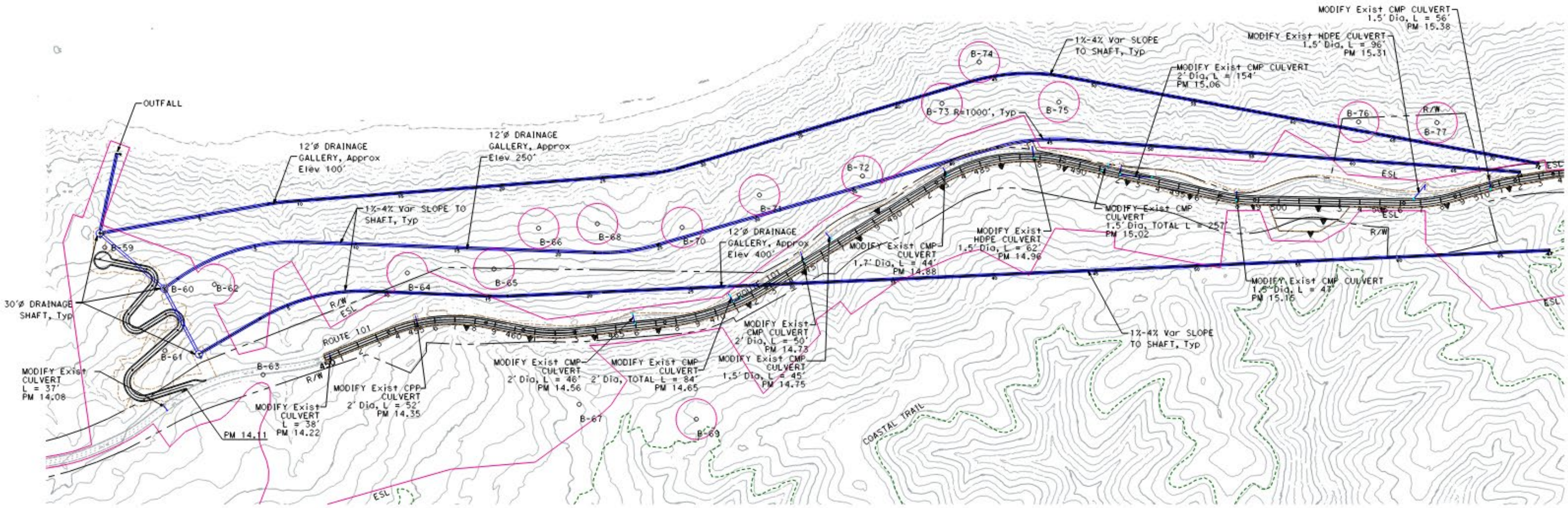


Figure 21. Underground Drainage System Layout

The purpose of the underground drainage system would be to drain the ground passively by gravity, reducing the groundwater level within the hillside that incorporates U.S. 101 and thereby reducing the pore pressures acting on the various landslide failure surfaces. The draining would increase the effective stress on these surfaces and increase the factors of safety against sliding. The effectiveness of the underground drainage system would depend in part on the hydrogeologic properties of the rock mass and the landslide deposits.

The drainage gallery tunnels would be arrayed below the lowest potential failure surfaces and distributed vertically to provide access for the drains to penetrate most of the overlying Broken Formation rock or rock/debris landslide deposits. As shown in Figure 22, the drainage gallery tunnels would be positioned at about elevations 90 feet, 180 feet, and 280 feet at the shaft locations. More detailed site investigations will be required to optimize locations and depths of the drainage gallery tunnels.

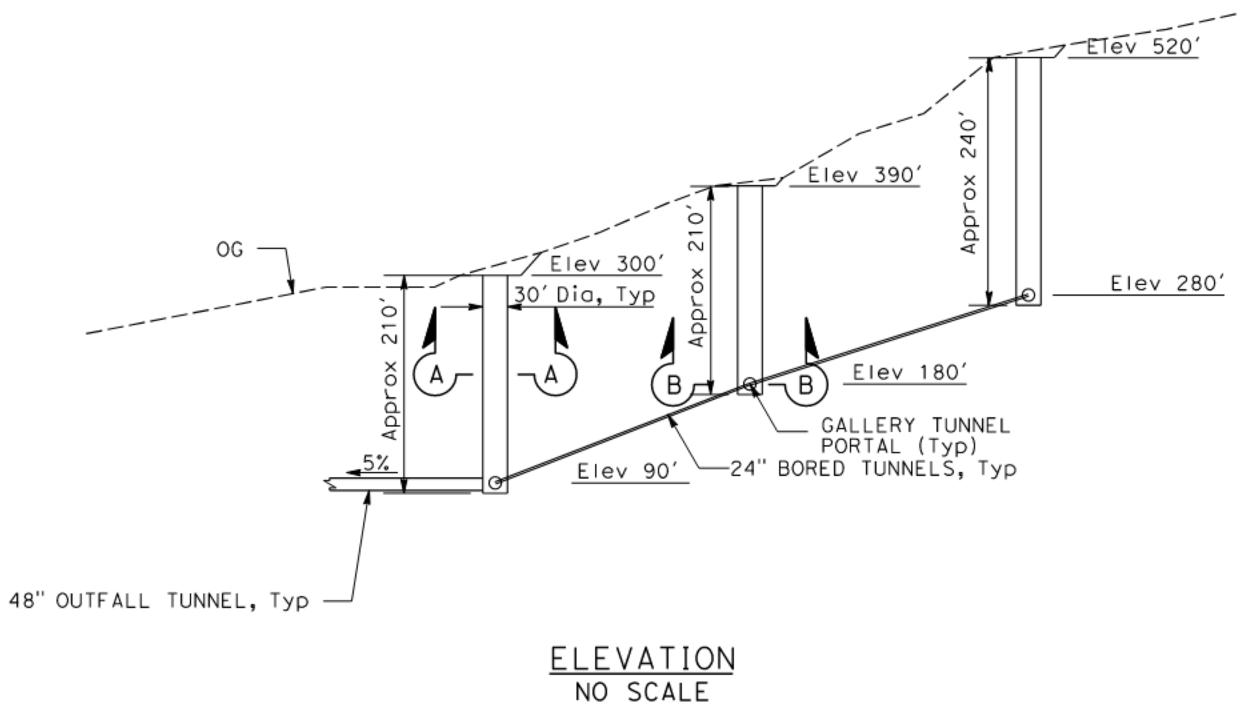


Figure 22. Underground Drainage System Elevation

Figure 23 presents a typical section of a drainage gallery tunnel. As shown, the drainage gallery tunnels would have outside diameters of approximately 12 feet. This diameter was selected to provide sufficient space for the small drilling equipment needed to install the drains. The drainage gallery tunnels would be excavated by TBM and lined with precast segments of reinforced concrete. Lengths of the tunnels would vary from 6,700 to 7,200 feet.

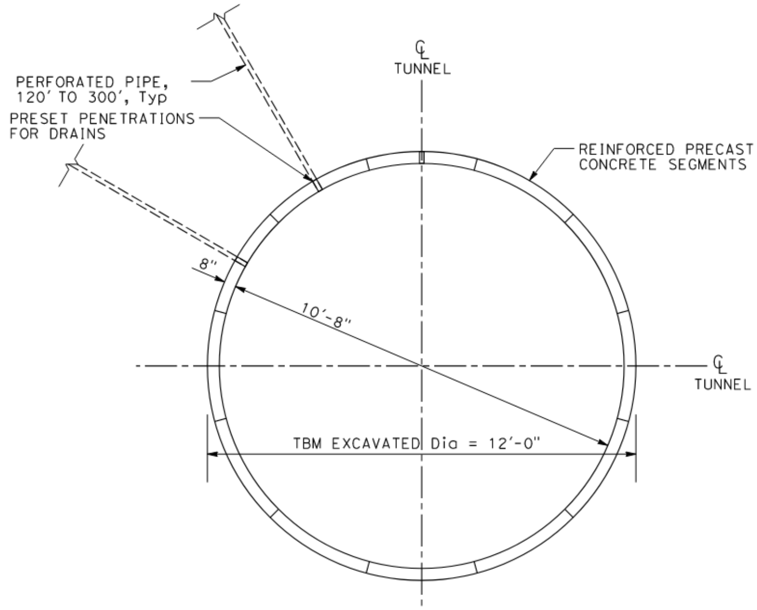


Figure 23. Drainage Gallery Tunnel Section

The drainage gallery tunnels would slope approximately 1 percent towards the shafts to ensure passive gravity flow. The shafts would be interconnected with 24-inch diameter drain bores which would lead to the lowest shaft which in turn would have a single 4-foot diameter bore leading to an outfall structure draining on to a rip-rap slope leading to the Pacific Ocean.

Perforated pipe drains would be radially installed from the drainage gallery tunnels and would drain into the galleries. To facilitate drain drilling, the precast concrete tunnel liner segments could be configured to provide pre-established drilling locations clear of the lining reinforcement. The drains would be drilled through these ports at preset orientations to intersect the various failure surface planes and water-bearing layers. More detailed site investigations would be required to establish optimal spacing, length, and orientation of the drains.

The three vertical shafts at the south end of the underground drainage system would have inside diameters of about 30 feet, as shown in Figure 24. The shafts would vary in depth from about 210 to 240 feet (Figure 22). Upon completion of construction, the shafts would provide maintenance access to the gallery tunnels.

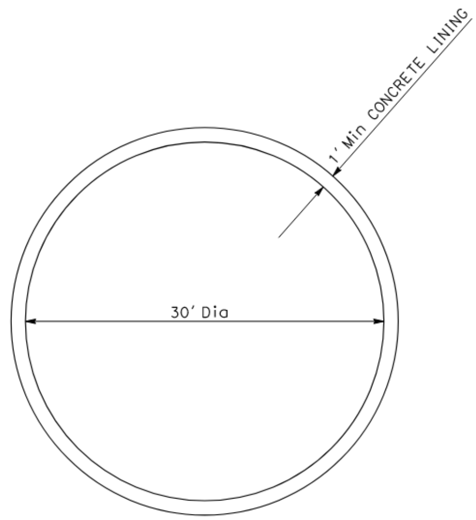


Figure 24. Drainage Shaft Section

The shaft linings would consist of an initial flashcrete lining, followed by either a permanent CIP or shotcrete lining. The 12-inch-thick lining would and be designed to allow for water pressure relief. Temporary rock bolts to stabilize rock wedges could be installed if indicated by further geotechnical investigations or during construction.

- *Analysis and Results*

One cross section was developed for each landslide complex to evaluate the stability of the slope for the existing and dewatered conditions. The cross sections are transverse to the existing road and generally parallel to the direction of landslide movement. As indicated in Table 8, strength parameters were estimated from existing data and back analysis using the 2D slope stability analysis program, Slide2, by Rocscience Inc. (Build 9.018, 2021b).

For the non-earthflow landslide areas, the slope stability models considered a failure surface at a depth that is consistent with the inclinometer data. For the earthflow landslide area, the landslide consists of Melange rocks comprised of isolated, detached rock blocks entrained within a highly sheared siltstone or argillite matrix. It is likely that there exist multiple basal slide surfaces rather than a single, continuous basal slide surface in the Melange rocks. Therefore, the slope stability failure search was not constrained to the failure surface. The internal friction angle of the failure surfaces was calibrated until the factor of safety was equal to about 1.0.

Coastal erosion was modeled by removing landslide material from the landslide toe slope. The amount of material removal was approximately determined by translating the existing toe slope landward to the nearest slope break. The length of removal is provided in the notes of Table 25.

The results of the static and pseudostatic slope stability analyses for the existing and dewatered conditions are presented in Table 25 and Appendix A.

Table 25. Slope Stability Analysis Results

			Earthflow Landslide	**Rock/Debris Landslide**		
		Section	**1 (EF)**	**2 (WC)**	**3 (SLCG)**	**4 (NLCG)**
Case			**Factor of Safety (Janbu Simplified / Spencer / Bishop Simplified) (1)**			
Existing Condition	Static	Existing Groundwater	0.89 / 0.92 (4)	0.99 / 1.01	0.98 / 1.01	1.00 / 1.53 / 1.02
		Dewatered	1.05 / 1.10	1.07 / 1.09	1.14 / 1.19	- (6)
	Pseudostatic (2)	Existing Groundwater	0.55 / 0.58	0.55 / 0.57	0.53 / 0.57	0.57 / 0.92 / 0.59
		Dewatered	0.61 / 0.68	0.60 / 0.62	0.66 / 0.70	- (6)
Coastal Erosion (3)	Static	Existing Groundwater	0.95 / 1.00 (4)(5)	0.98 / 1.00	0.98 / 1.01	0.97 / 1.00
		Dewatered	0.98 / 1.03 (4)	1.04 / 1.08	1.17 / 1.20 (5)	- (6)
	Pseudostatic (2)	Existing Groundwater	0.54 / 0.58	0.54 / 0.56	0.53 / 0.56	0.56 / 0.58
		Dewatered	0.59 / 0.64	0.60 / 0.63	0.66 / 0.70	- (6)

Notes:

(1) Factors of Safety were computed using a non-circular failure search and the Janbu simplified and Spencer methods. Where Factors of Safety estimated from Janbu Simplified and Spencer methods were less comparable, Bishop simplified method was also used.

(2) A horizontal seismic load, k_h, of ⅓ PGA=0.29 was applied to the pseudostatic case.

(3) Landslide material was removed from the landslide toe slope at the following approximate distances behind the existing landslide toe to simulate coastal erosion:
Section 1 – 46 feet, Section 2 – 48 feet, Section 3 – 29 feet, and Section 4 – 44 feet.

(4) Failure surfaces with a Factor of Safety of less than 1.0 are localized and occur within approximately 70 feet of the existing landslide toe.

(5) The critical slip surface entry point for the coastal erosion condition is upslope of the entry point for the existing condition, resulting in a Factor of Safety greater than that of the existing condition.

(6) Groundwater was not encountered above the landslide failure zone during Phase 2B investigations.

- *Evaluation*

Based on the slope stability analysis for the four landslide complexes, the increase in the overall factor of safety from underground drainage was up to 0.2. Additional groundwater and geotechnical data would be needed to further evaluate effectiveness and impacts of the underground drainage system concept.

Long-term maintenance of the underground drainage system would include groundwater level monitoring, water quality monitoring, tunnel inspection, and drain cleanouts. The three shafts of the underground drainage system would be located in an area of the earthflow where westward creep of about 1 to 2 inches per year has been observed. The shafts either would need to be periodically repaired or would need to be designed with collapsible columns to accommodate earthflow movements. It is also anticipated that even with the underground drainage system in place, there could be further rock/debris landslide movements above the drainage gallery tunnels. Therefore, it is anticipated that over time the radial drains of the underground drainage system would need to be redrilled to function as designed.

4.1.2 Construction Considerations

The current design alignment for Alternative X includes retaining wall structures. The wall structures would be constructed within landslide complexes of the Broken Formation which is composed primarily of massive to thickly bedded hard to very hard sandstone. Excavations in this material would likely require specialized equipment and may require blasting in some areas. Conversely, weak, adversely dipping bedrock structures may be encountered that are currently unknown at this time, potentially requiring remedial grading measures for stabilization beyond the original design configuration. Groundwater data suggest that the regional groundwater level within the Broken Formation is deeper than the design components. However, this data also suggests the groundwater regime is complex and controlled by fracture flow with significant head pressures. As such, sporadic zones of fracture flow with potentially significant seepage may be encountered at shallower depths during earthwork operations and should be anticipated during construction.

For construction of the underground drainage system, small diameter rock TBMs would be best for drainage gallery tunnel construction. The TBMs would be launched from the three shafts. The shafts would serve for materials delivery, muck removal, and long-term access for maintenance.

The inferred poor quality of the rock along the drainage gallery tunnel alignments is not conducive to the use of traditional side-grippers to achieve the reaction needed for the TBM thrust. A more reliable means of achieving the required thrust would be to use precast liner segments for the required reaction. A shielded TBM design is considered suitable for the anticipated ground conditions featuring fractured rock with water-bearing zones. The cutter head design would require more specialized material testing to assess potential wear and drillability.

Instead of constructing reception shafts to remove the TBMs used for construction of the drainage gallery tunnels, the TBMs would be skeletonized and abandoned at the end of each tunnel drive. This strategy would eliminate the need to excavate costly retrieval shafts and associated muck handling and disposal issues.

The relatively high quartz content of the sandstone along the alignment indicates that a portion of the intact rock and the rock fragments to be encountered in drainage gallery tunnel excavations could be abrasive. If confirmed by additional petrographic analyses and abrasiveness testing, this could result in accelerated wear on TBM cutters as well as muck handling equipment, which would need to be considered in planning and scheduling.

The drainage shafts would be constructed in a sloping and space-constricted area that would limit the type of construction that could be performed. The shafts would be sunk or advanced using either drill-and-blast excavation methods or modified roadheader equipment. The productivity of the excavation may be constrained by the limitations on truck movements along U.S. 101.

The underground work area will require ventilation both during construction and for longer-term routine inspections and maintenance.

A site for disposal of excavated material in proximity to the site will need to be identified.

4.2 Alternative F

4.2.1 Evaluation of Alternative F Design Components

The conceptual design for Alternative F current as of October 26, 2023 was developed based on limited subsurface information. Currently, there are only six borings along the alignment located near the North and South Portals and the OMC, and assumption on the subsurface conditions between these borings were made. The key assumptions were:

1. The permeability of the earthflow landslide deposits is consistent with that of clay.
2. Average earthflow movement is 2 inches per year, with predominant movement downslope and perpendicular to current slope contours.
3. If the crown of the tunnel can be maintained at least 20 to 40 feet below the basal failure zone, effects of landsliding on the tunnel will be minimal.
4. The tunnels will either pass through the Broken Formation where the profile is deep (>150 feet) or through the Melange where the profile is shallower (<150 feet).
5. The North Portal conditions consist of limited soil cover (<60 feet) of colluvium and rock landslide debris overlying the Broken Formation.
6. The tunnel alignment is sufficiently east of the basal failure zones extending from the shore slope and daylighting east of the U.S. 101.

4.2.1.1 South Portal Cut-and-Cover Approach with EDAS

- *Description*

The South Portal area is situated within the earthflow, as shown in the geologic map, Plate 6. The South Portal approach cut-and-cover section, which includes walls RW 2R and RW 2L, would be a structure serving to not only retain the adjoining ground materials, but also to manage ongoing earthflow movement.

As shown in Figure 25, the design concept uses large-diameter secant piles and engineered deformation absorption columns to absorb the earthflow ground movements. The secant piles would be socketed into the Melange well below the earthflow basal failure surface. The EDAS would extend through the zone of earthflow movement, and the system's strength would be engineered to slightly exceed the existing earth pressures in the earthflow.

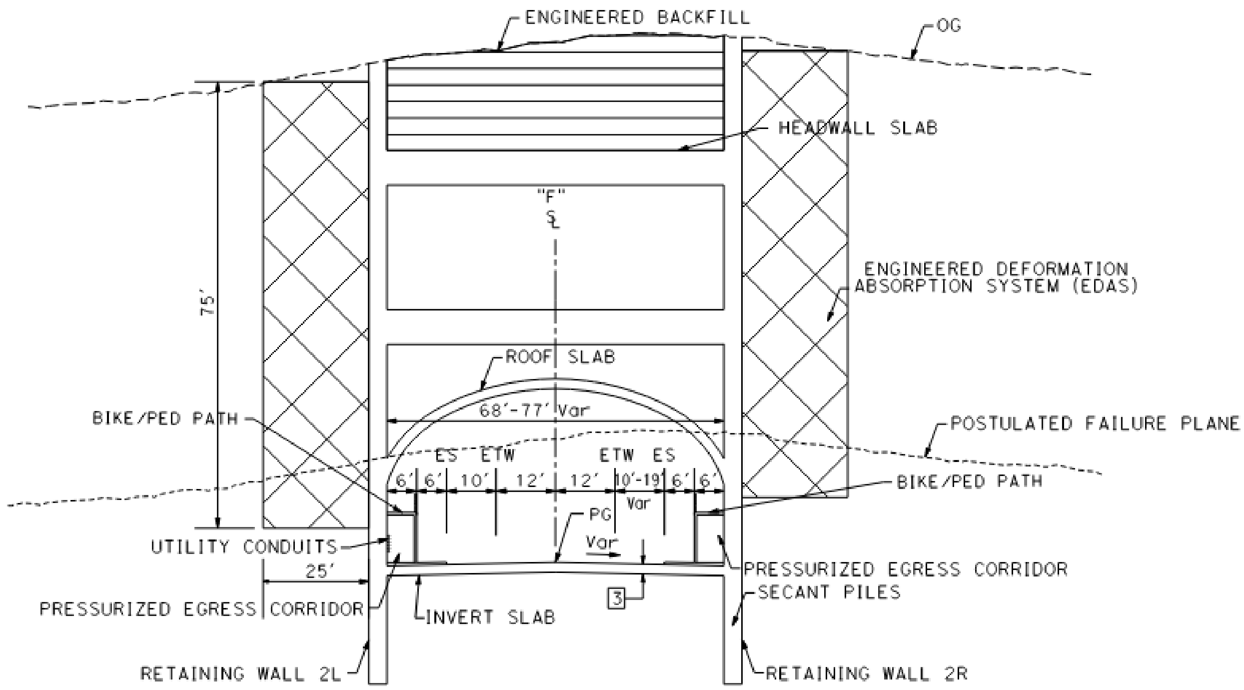


Figure 25. South Portal Cut-and-Cover Approach EDAS

The areal extent of the EDAS would be established to absorb the downslope moven earthflow. These include the currently observed downslope creep and the lateral anticipated to occur as a result of the design seismic event. Lateral support of th structure walls would be provided by interior slabs within the approach structure, as shown in Figure 25.

The earthflow would bear on the approach structure headwall. The upper approximate 75 feet of the headwall will be subjected to this load. Since this load is displacement derived, (e.g., earthflow downslope creep), the design approach absorbs this deflection using the EDAS. The strength of the deformable columns would be the limiting loads on the approach structure. Because the approach structure is not perfectly aligned with the downslope movement, the EDAS would also be provided along the sides of the structure to limit those loads as well.

- *Analysis and Results*

Further study is needed to determine if a deformation absorption system such as the EDAS proposed for the South Portal approach has ever been used to isolate a transportation tunnel from active landslide loading. The loading that would be imposed on a rigid south portal approach structure embedded in the earthflow materials would be high due to the engagement of earthflow materials on either side of the approach structure. The frictional properties of soil cause the anchoring effects of an immovable object to engage not only the soil immediately upslope from the object, but also volumes of adjacent soils. There is an opposite effect when the object is flexible. However, as a flexible object deflects, the upslope soils arch around the object, shedding the load to the soils on either side. This “trap door” effect is illustrated in Figure 26.

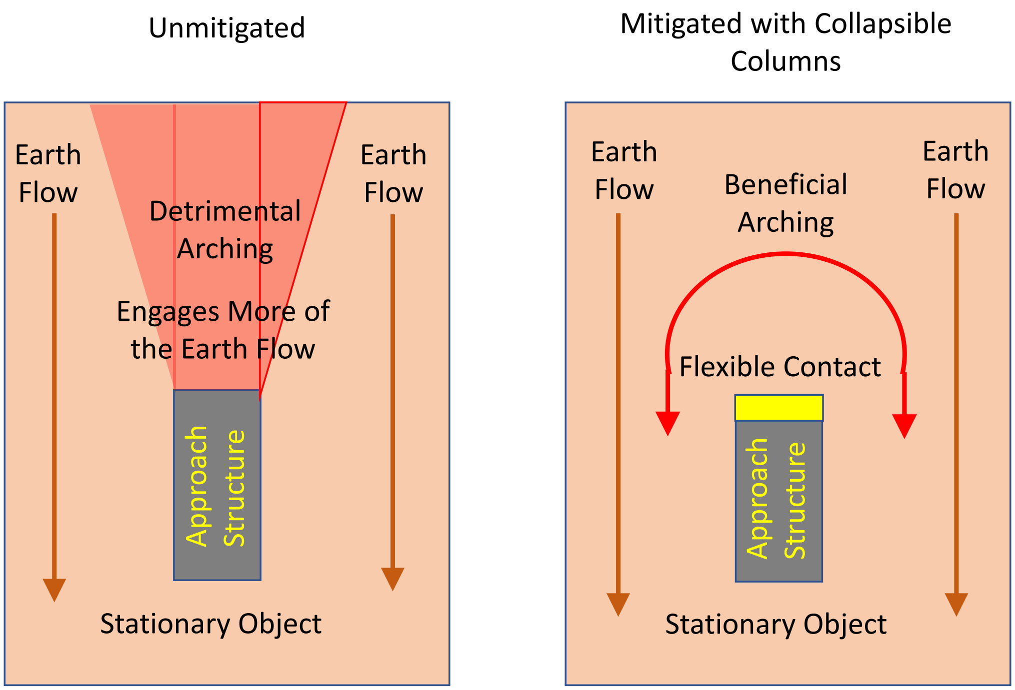


Figure 26. Earthflow Loading

The structural concept was modelled using MIDAS GTS and cross checked with hand calculations. The results indicate that the loads imposed on the structure can be prescribed and effectively transmitted to the portions of the secant piles embedded (keyed) in the Melange. In addition, the stress levels in the Melange and the corresponding deflections are well within acceptable limits. Figure 27 is a half-section cut away of the MIDAS model. Analyses are presented in Appendix C.

The LCG earthflow has a history of both lateral and vertical motion occurring in seemingly random locations and times. For this study, it was assumed that the average movement is 2 inches per year. The earthflow’s predominant movement is down slope and roughly perpendicular to the slope contours. At the Alternative F south portal approach structure the motion is southwest and approximately aligns with the centerline axis of the cut-and-cover section approach.

The earthflow materials consist of decomposed sandstone and argillite with properties like a stiff sandy clay with blocks of intact rock (Caltrans, 2022a). The depth of the earthflow’s basal failure surface is approximately 75 feet at the Alternative F South Portal approach structure location. The limited data available indicates that the water level may be a few feet above the earthflow basal failure surface.

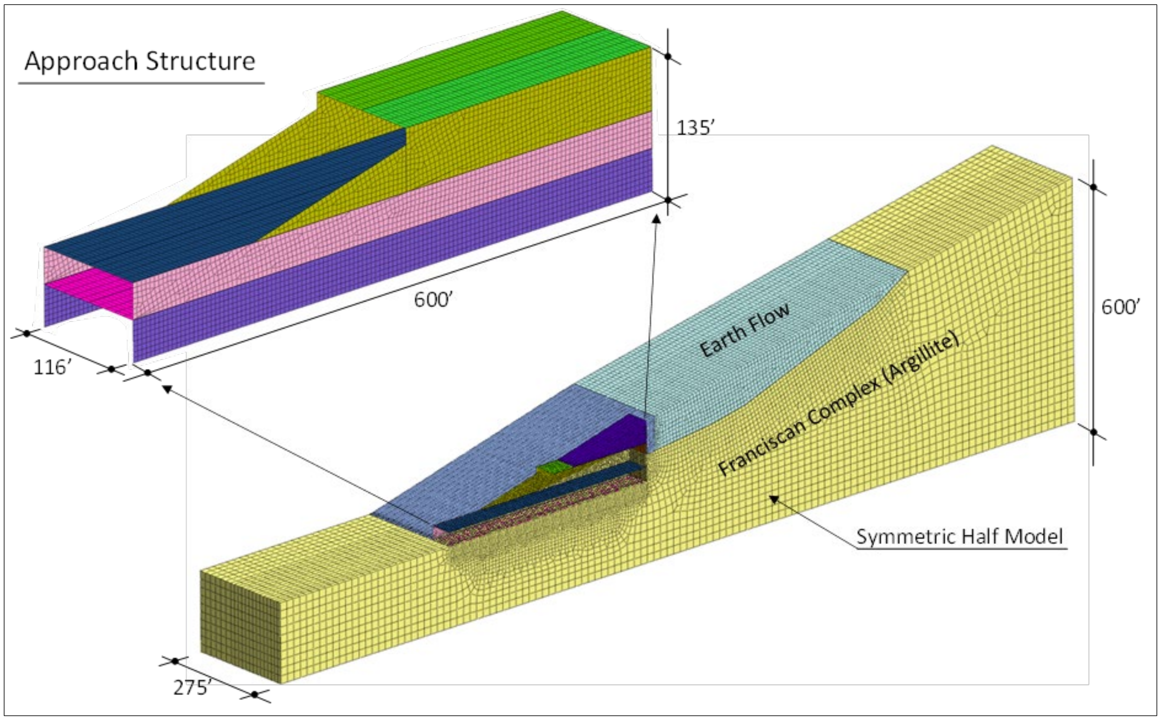


Figure 27. MIDAS 3D Model

To establish the collapse strength of the columns it was necessary to determine the current earth pressures in the surrounding earthflow soils. Theoretical calculations suggest the corresponding earth pressure coefficient should be approximately 0.74, based on a Coulomb analysis. In-situ pressuremeter tests taken in the earthflow (Caltrans, 2022a) indicate earth pressure coefficients in the range of 0.7 to 0.85. This close correlation between theory and practice provides confirmation and a reasonable level of confidence in the selection of the column strength criteria. For the purposes of this study an earth pressure coefficient of 0.8 was used, and a stepped strength profile was used for modeling, as shown in Figure 28.

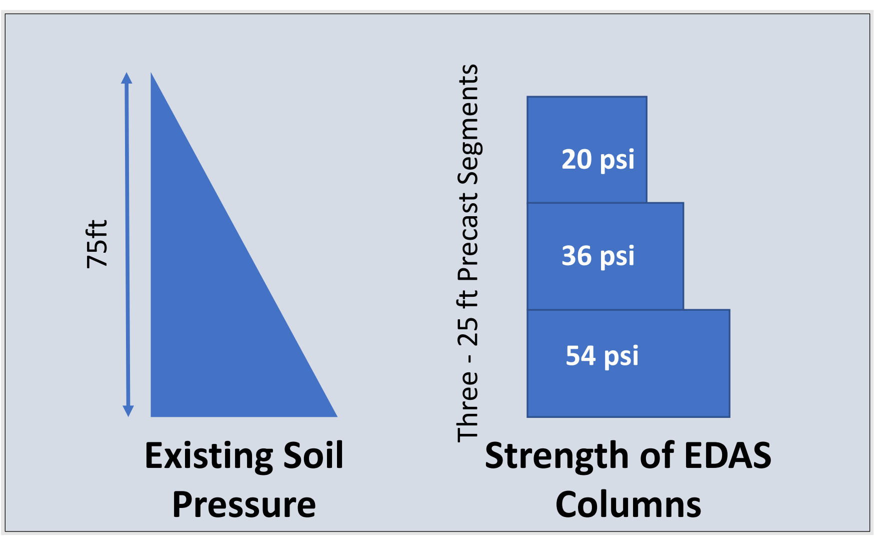


Figure 28. Stepped EDAS Strength Profile

The required width of the EDAS treatment zone is dependent upon the projected downslope movement of the earthflow over the life of the structure. This movement was established by using an estimated yearly down slope movement that was extended over the service life of the tunnel and the estimated lateral spreading anticipated to occur due to the design seismic event.

For the ongoing downslope creep, a rate of 2 inches per year for a minimum of 75-year service life was assumed. This translates into a deflection of 12.5 feet. In addition to this deflection, an additional downslope movement of 22.8 feet would be accommodated for seismic lateral spreading. Estimates of lateral spreading were empirically derived, and the results differ significantly depending on what method is used. Table 26 shows the range of these estimated deflections, which range from 4 to 22.8 feet. Considering these two modes of earthflow movement, an EDAS width of 35 feet was selected. More in-depth analyses should be performed in subsequent stages of the design.

Table 26. Earthflow Lateral Spreading Estimates

Method	Displacement (ft)
Bray and Travasarou (2007)	4
Rathje and Saygli (2009)	22.8
Jibson (2007)	6.5

Note:
M=8.8, PGA 0.88g, a_{yield} =0.1g

- *Evaluation*

By providing a zone of crushable material along the exposed sides of the south portal approach structure, loads imposed by continued downslope earthflow movement would be limited to the strength of the collapsible columns. As the earthflow migrates downslope, the columns would be progressively crushed to absorb the motion.

Although assumptions for deflection and lateral spreading are reasonable, if yearly deflections exceed predictions, or if the service life should be extended, additional columns could be added to extend the functional life of the structure.

Further study is required to establish the handling and placement requirements for the column sections. The collapsible columns would be prefabricated, transported to the site and the inserted in pre-drilled holes. The treatment depth would be to the top of the earthflow failure surface or approximately 75 feet. The columns would be pe-fabricated in shorter sections, say 25 feet, and their strengths “tuned” to the corresponding earth pressure. Due to the nature of collapsible concrete, the prefabricated columns will have to be cast in a horizonal orientation to prevent collapse of the foam concrete under its own weight. The column segments would then be lowered into a pre-drilled hole with any annular space grouting to ensure contact with the surrounding soils.

The South Portal cut-and-cover approach with EDAS would allow portal construction in an active earthflow area and so allow shortening of the Alternative F tunnel alignment. It would also reduce the length of SEM tunneling required in the relatively unfavorable ground conditions of the

Franciscan Complex Melange.

4.2.1.2 SEM Tunnel

- *Description*

As discussed in the Alternative F description in Section 1.6, the tunnel would be configured for two-way traffic and would be approximately 6,000 feet long. It would be sized to provide truck height (16 feet, 6 inches) clearance for two 12-foot-wide travel lanes and two 10-foot-wide shoulders (Figure 29). There would be two emergency corridors on either side, and the roofs of these corridors would be bike lanes. The tunnel's interior spring line width is 66.25 feet, and the floor to ceiling height is 35 feet.

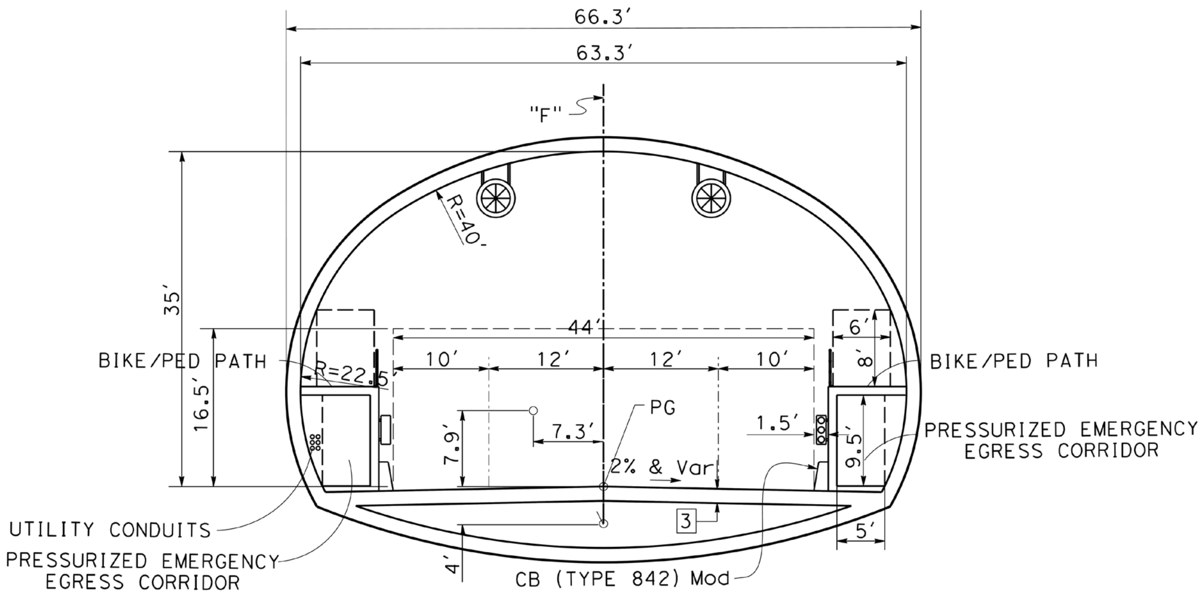


Figure 29. SEM Tunnel Section

The Alternative F tunnel alignment was established to commence as far north along U.S. 101 as possible, while staying below the basal failure surfaces of the LCG landslide complexes. This alignment is aligned vertically at the South Portal to pass approximately 25 feet below the base of the earthflow and would requires construction of an approach structure within the earthflow as described in Section 4.2.1.1.

- *Analysis and Results*

The ground conditions anticipated along the Alternative F tunnel alignment consist of Franciscan Melange and Broken Formation. Both materials can be mined using SEM tunneling. Due to its soil-like nature, the Mélange would have to be mined using multiple heading segments, as shown in Figure 30.

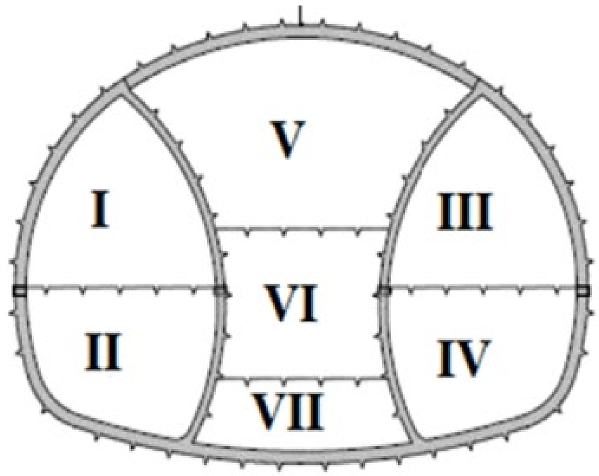


Figure 30. Sequential Excavation

Depending on further subsurface investigation findings for the Melange in Reach 3, the SEM process may require ceiling pre-support to control roof raveling.

SEM mining in the Broken Formation in Reach 4 would also require multiple heading segments but probably fewer than in the Melange. The combination of the fractured rock conditions and the considerable width of the two-lane tunnel would require relatively closely spaced rock bolts to create a contiguous rock arch over the tunnel excavation.

Groundwater data obtained to date do not indicate high water pressure conditions along the alignment. These conditions would require further study in the next phase of subsurface investigation.

Considering the challenging ground conditions and the seismic environment, the seismic performance of the tunnel structure was evaluated as a fatal flaw check. A series of finite element analyses was performed to determine the structural requirements needed for the tunnel to survive the design seismic event.

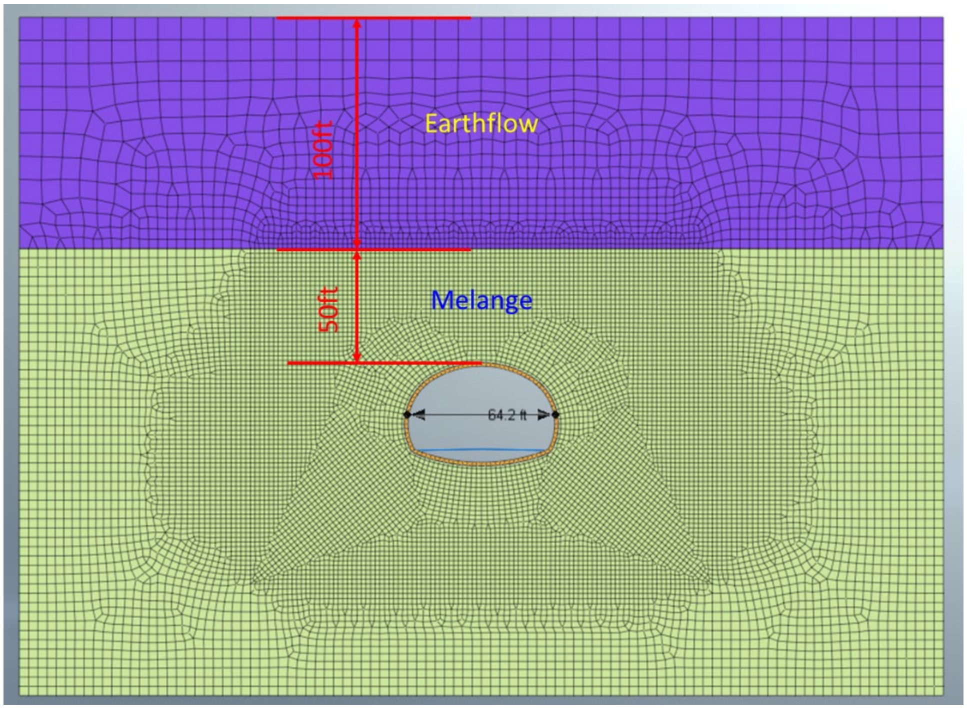
A set of geologic cross sections and the geologic profile developed across and along the proposed Alternative F alignment were used to develop interpretations of potential landslide geometries to evaluate landslide scenarios that could impact the proposed tunnel alignment.

Finite element analyses using MIDAS GTS were performed to estimate the seismically induced stresses and strains in the tunnel lining. The model incorporates vertical and horizontal loading as well as ground shaking. The derived values were then compared to stress and strain levels that have been shown to be acceptable in concrete linings.

These analyses show that the large, two-lane sequentially excavated tunnel in the Melange would require a robust lining system to survive the design earthquake. Where under more normal loading conditions the lining would be on the order of 16 to 18 inches thick, this tunnel would require a lining thickness of 24 inches. Analysis details are included in Appendix C.

The modes of failure for a tunnel undergoing ground shaking are excessive strains in the lining due to ground-induced shear distortion and overstressing of the lining materials. The preliminary analyses indicated that strains and stresses induced in the proposed lining, currently sized at

24 inches thick, are well within acceptable limits. Figure 31 and Figure 32 illustrate the stress regime for two cases, a shallow rock case (Reach 3) and a deep rock case (Reach 4). These cases bound the tunnel profile conditions along the alignment. In both cases, the calculated stresses are well within the interaction diagrams meeting the criteria established by ACI-318, shown by blue lines in the figures. These analyses will need to be revisited in more detail as additional subsurface information becomes available and more in-depth modeling can be performed.



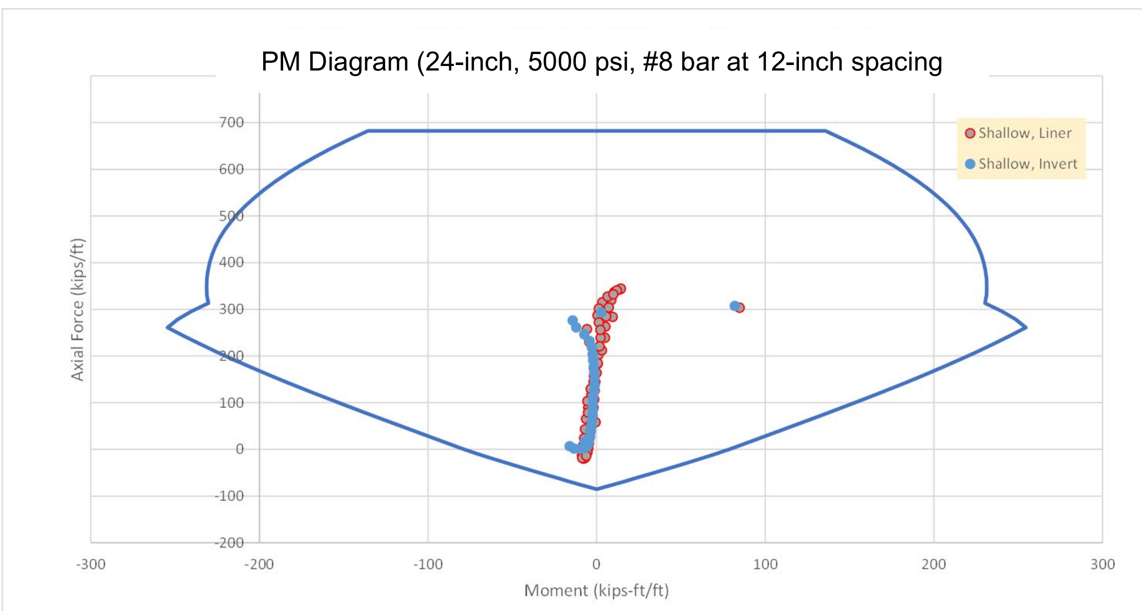


Figure 31. SEM Tunnel in Shallow Rock Cover Condition

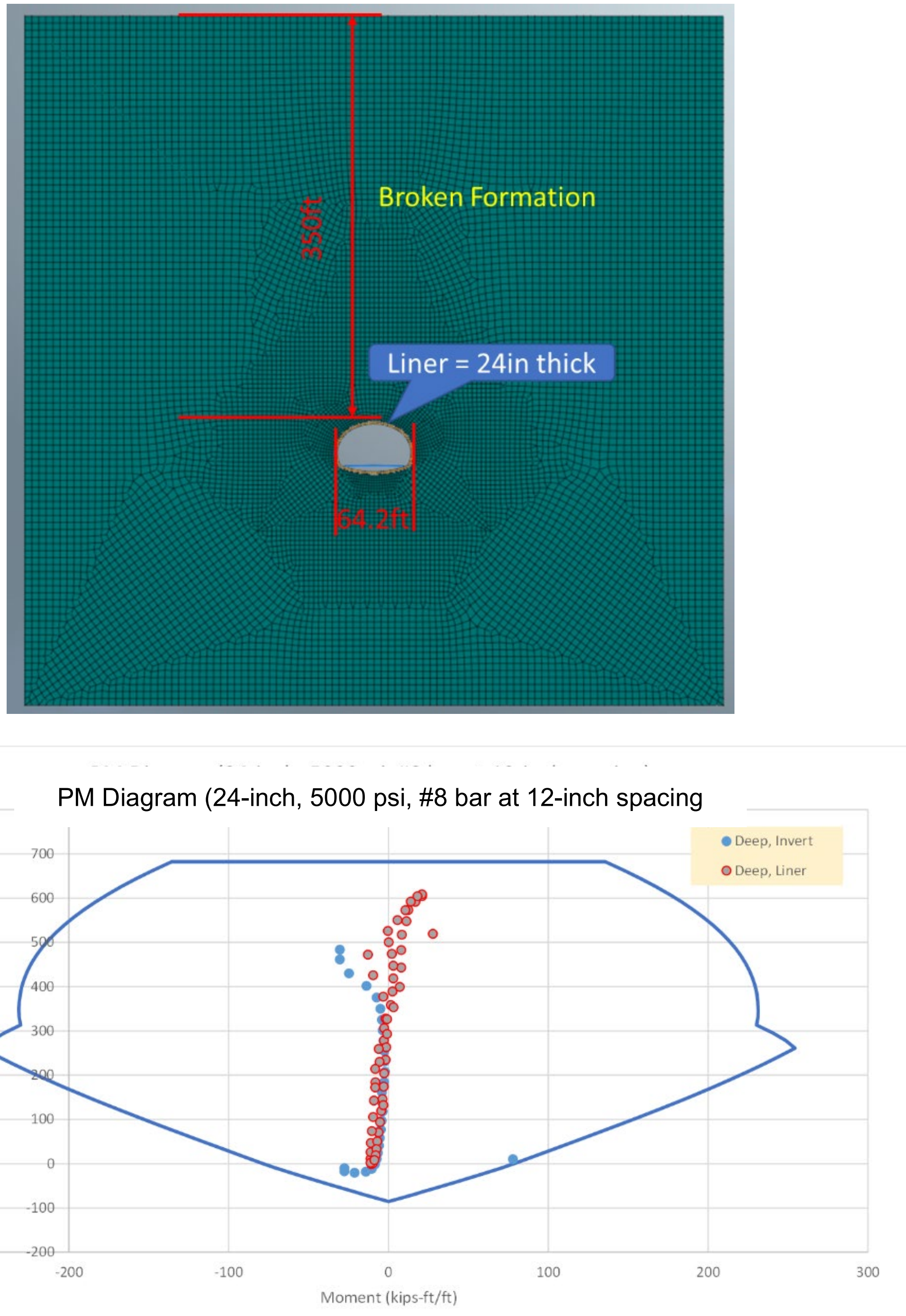


Figure 32. SEM Tunnel in Deep Rock Cover Condition

- *Evaluation*

The analyses indicated that the design and construction of a large-diameter SEM tunnel is feasible in the ground conditions anticipated for Alternative F.

The FEM analyses indicated that the concept of an SEM is valid from the perspective of seismic ground loading and seismic ground shaking.

4.2.1.3 North Portal and Bridge Approach

- *Description*

As discussed in the project description in Section 1.6, the North Portal headwall and immediate rock slopes would be supported by permanent rock bolts and CIP facias, while the portal approach would be supported by retaining walls (RW 3R/3L) which are anticipated to be CIDH piles and lagging with permanent ground anchors (Figure 33). These retaining walls would be up to 40 feet high and would be at the south end of the Wilson Creek Tributary Bridge connecting the portal headwall to U.S. 101.

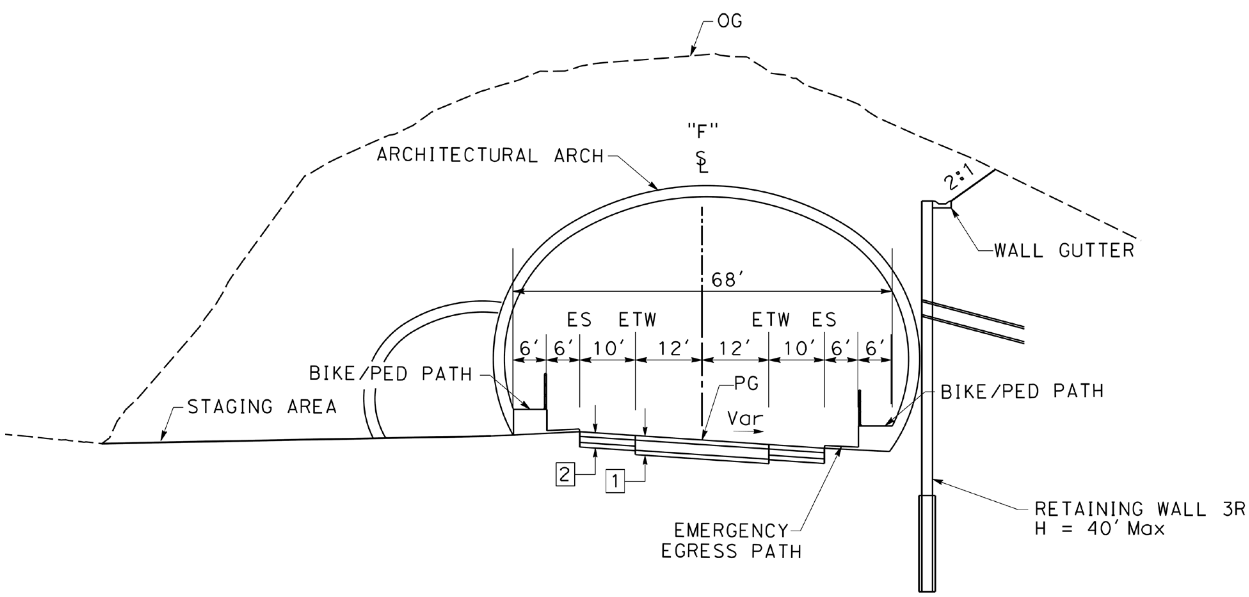


Figure 33. North Portal Approach

The North Portal headwall would include an architectural arch on the north-sloping rock face. The final portal structure would consist of CIP concrete that is context-sensitive, tinted and textured to blend into the existing geology. A representative example of a context-sensitive portal design is shown in Figure 34. Rockfall protection features, such as a canopy portal extension, would be included at the North Portal.

Figure 34. Context-Sensitive Portal Design

As discussed in Section 1.6, the proposed Wilson Creek Tributary Bridge at the North Portal would be a single-span, precast concrete I-girder with a CIP concrete deck (Figure 35). The bridge would have a total structure length of 122 feet. The bridge would carry one 12-foot-wide traffic lane in each northbound and southbound direction with two 10-foot-wide shoulders alongside each traffic lane. The proposed abutments are seat-type abutments founded on 24-inch diameter CIDH reinforced concrete piles.

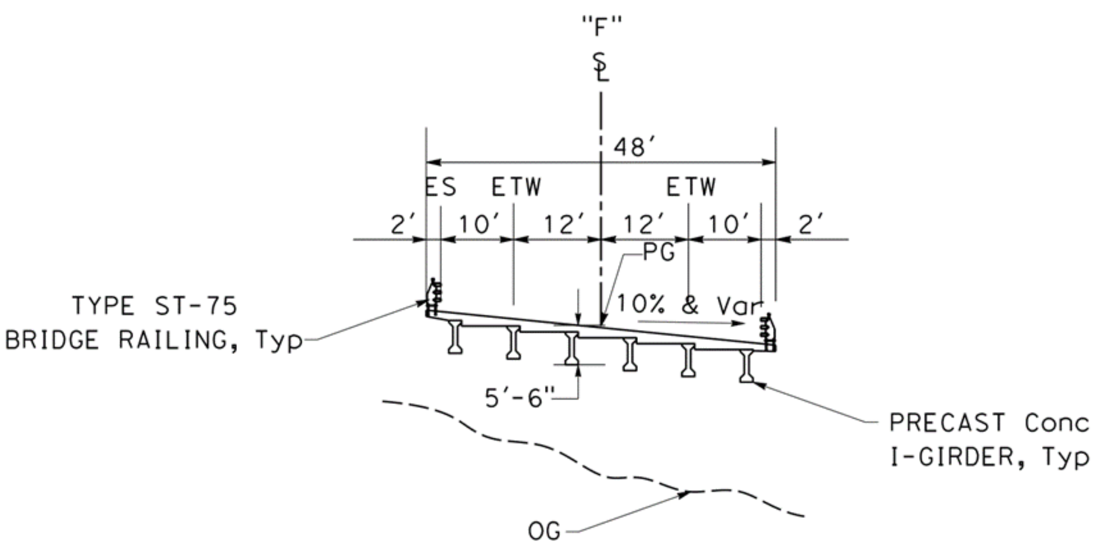
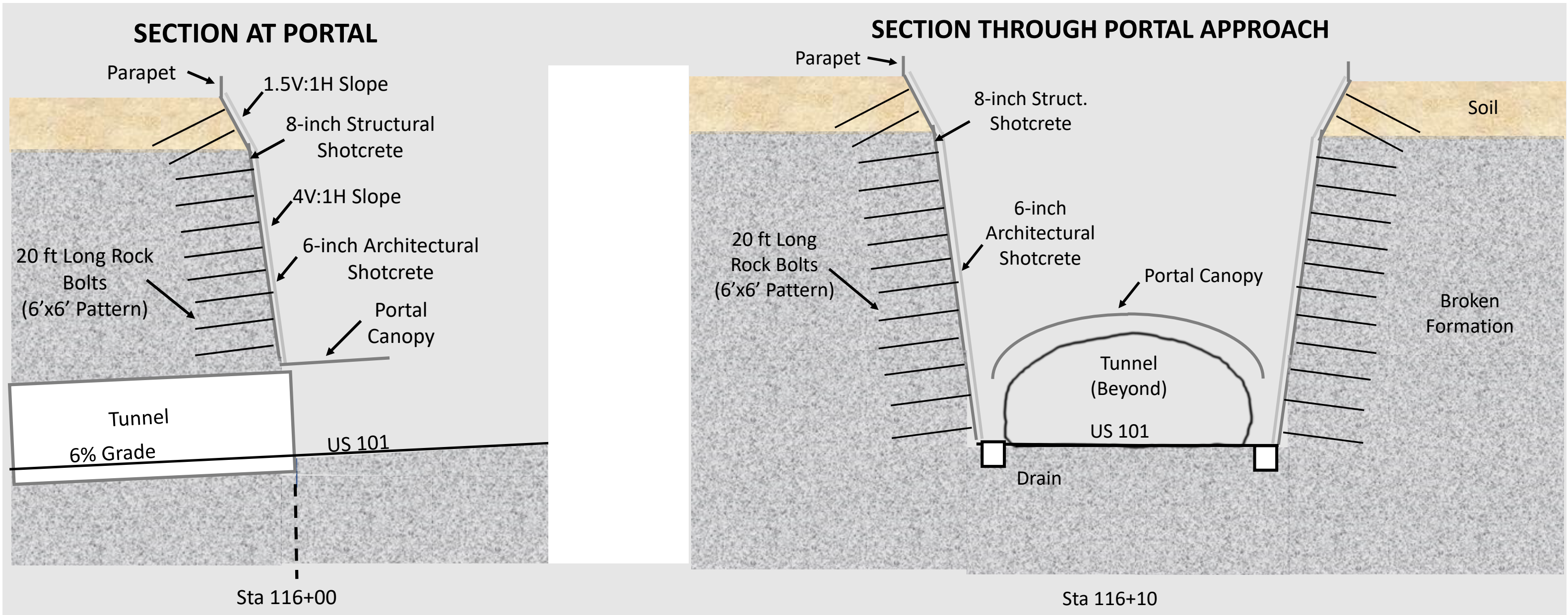


Figure 35. Wilson Creek Tributary Bridge

The Wilson Creek Tributary Bridge would be located just east of the main head scarp of the NLCG, where a thin layer of Colluvium overlies variably weathered rock of the Broken Formation. The bridge would span across a southwest-to-northeast oriented ravine that extends northeastward toward Wilson Creek. Surface water is anticipated to flow generally in a northeasterly direction down the ravine that ultimately leads to Wilson Creek to the east.

- *Analysis and Results*

The current conditions at the North Portal are not known in sufficient detail to determine the precise slope stabilization that will be needed. However, considering the existing slope and the presence of colluvium and fractured Ground Class II Broken Formation rock, measures would be needed to stabilize the slopes above and adjacent to the portal, as shown in Figure 36. These measures may include the removal of loose rock materials, regrading, rock bolting of unstable rock wedges, and revegetation. Additional subsurface investigations would be required to determine the extent of the required stabilization.



Elevation View **Section View**

Figure 36. North Portal Stabilization

Construction of the North Portal approach would require clearing and grubbing of the impacted rock face. With the rock face exposed, the portal excavation would commence at the portal entrance. The rock face surrounding the portal excavation would likely require temporary stabilization due to its high degree of fracturing. This temporary stabilization would be a combination of rock bolts and shotcrete, and it would provide stabilization support until the final portal structure is completed.

Beyond the rock stabilization at the tunnel portal, the planned retaining walls would limit the excavation volume and surface impacts. Drilled-in-pile and lagging walls would be used to retain the rock slopes, and the permanent ground anchors used in conjunction with socketed soldier beams would provide the required lateral support.

The seismic demand for the North Portal approach Wilson Tributary Bridge is expected to be very high, as indicated by the preliminary PGA of 0.85g. There is no liquefaction potential. Bedrock or dense material is expected to be at a depth of about 5 to 15 feet.

- *Evaluation*

In the absence of site-specific geotechnical information at the North Portal, the evaluation is general in nature and will be updated as information becomes available. However, North Portal design and construction considers these known site conditions.

For the assumed Ground Class II quality of the Broken Formation bedrock, the design includes pre-support of the initial opening periphery and canopy arch pipe installed horizontally from the headwall.

For stabilization of the headwall above and adjacent to the portal, the design includes installation of pattern soil nails and/or ground anchors with shotcrete surface protection. If groundwater control of the headwall face surrounding the portal opening is required, it would be achieved using designed wall drainage strips placed on horizontal and vertical grid and connected to weepholes at the base of the excavation.

For the need for face support and increased ground stand-up time, fiberglass spot dowels would be installed as needed to provide face support and increase stand-up time of the ground.

Foundation recommendations for the Wilson Creek Tributary Bridge and retaining walls are consistent with soil conditions, based on data from the limited site investigations.

4.2.1.4 Operations and Maintenance Center

- *Description*

The OMC will be located approximately 0.6 mile south of the tunnel South Portal, on 1.4 acres. The site would include a building, parking spaces, outdoor storage, and maintenance equipment.

The building would be an approximately 12-foot-tall, 18,000-square-foot, single-story structure. It would contain equipment and other facilities related to tunnel maintenance, operations and emergency response. It is anticipated the roof would be planted (i.e., a “green” roof) to blend into the surrounding terrain.

Retaining walls would be located around the OMC building and yard for security purposes and to provide a grade break that allows the OMC facilities to be placed below the existing ground surface. The site retaining walls are proposed to be constructed of reinforced concrete with heights up to 20 feet.

Construction of the OMC would involve cutting into the hillside and regrading a portion of the existing highway to create an access road to the facility. It is anticipated that porous pavement would be used to filter stormwater. The building sanitary sewer system would follow traditional plumbing methods, but it would discharge to a 3,000-gallon septic holding tank.

- *Analysis and Results*

The building foundation loads are anticipated to be relatively low. Based on available information, the preliminary assumption for groundwater table depth is 5 to 12 feet below ground surface. There is no liquefaction potential, and the site is located within the earthflow complex. Proposed foundation system shall be designed to maintain integrity of the supporting structure under a ground movement scenario, in order to prevent total collapse and maintain life safety.

The structures associated with the OMC have not yet been designed; however, with the exception of the water storage tank, their loading should be compatible with spread footing foundations. Site grades support gravity flows for sanitary piping from the building to the septic holding tank. Facilities should be designed to accommodate some westward movement using flexible connections.

According to the soil data from the existing Boring RC-18-001, the proposed foundations will be placed on gravelly silt, silty sand with gravel, or gravelly lean clay (colluvium).

- *Evaluation*

It should be noted that these recommendations are based on limited soil data and may be modified and revised once additional soil data becomes available.

Due to deep-seated nature of the landslides at the site, a deep foundation system may not be the best alternative for this site. Rigid shallow foundations could provide better performance during ground movement and allow the structures to float over earthflow with less damage. The recommended foundation types for structure support are as follows:

- Post-tensioned Slabs: Stiff post-tensioned slabs can be used to support the proposed building structures. The slab shall provide adequate stiffness to allow the supporting buildings to move as a monolithic structure with the earthflow.
- Stiff Reinforced Mat Foundations: A thick reinforced mat foundation or a mat foundation with rigid grade beams is another feasible foundation type for the buildings.

Per Chapter 18 - Soils and Foundations of the 2019 CBC, minimum footing embedment depth is 12 inches. A presumptive allowable vertical foundation pressure of 1.5 ksf and allowable lateral bearing pressure of 100 psf/ft can be used for preliminary design of spread footings. Allowable coefficient of friction for lateral sliding resistance is 0.25. These values can be increased by one-third when used with the alternative basic load combinations of Section 1605.3.2 of CBC 2019 that include wind or earthquake loads.

Preliminary maximum total settlement is estimated to be 2 inches, and the differential settlement can be assumed to be 50 percent of the total settlement. A modulus of subgrade reaction of 100 psi may be used for preliminary design of slabs. This value shall be adjusted for the size of the loaded area.

Due to the limited soil data, presence of expansive soils beneath the footings cannot be ruled out. If further investigation indicates that expansive soils are present, the slabs should be designed for appropriate uplift pressure due to soil expansion.

Because of the expected settlement, differential settlement and horizontal movement at the subject site, flexible joints are recommended in all conduits for the OMC buildings and equipment.

Cut slopes up to 2H:1V gradient can be used for site grading. Slopes should be properly benched, and appropriate drainage and erosion control measures should be provided to prevent erosion and sloughing. The recommendations of Section 1808.7 of CBC 2019 regarding footing setback from descending slopes and clearance from ascending slopes should be followed for building structures.

Due to the existing ground slope, site grading and retaining walls would be utilized to achieve flat building pads for the structures. Retaining walls would be located around the OMC building and yard for security purposes and to provide a grade break that allows the OMC facilities to be placed below the existing ground surface. The site retaining walls are proposed to be constructed of reinforced concrete with heights up to 20 feet. Based on these assumptions, concrete cantilever walls similar to Caltrans standard walls can be used. However, it should be noted that site PGA is larger than 0.6g, therefore standard plan walls may need to be modified. As an alternative, mechanically stabilized earth (MSE) retaining walls can be considered.

4.2.2 Construction Considerations

The following sections discuss geotechnical ground conditions to be considered for construction planning and execution for Alternative F if it is the selected alternative.

Subsurface conditions along the proposed Alternative F alignment are characteristically variable, and even with additional site explorations, conditions encountered during construction may differ from those based on exploration results. The primary sources of uncertainty are:

- The natural variability of the Franciscan Complex and the landslide deposits at the site.
- The unpredictable behavior of soil-like and rock-like IGM when excavated.
- The location of the landslide complex basal failure surfaces with respect to the tunnel alignment.
- Variable groundwater conditions due in part the natural variability of subsurface materials, confined, artesian, and possible perched water conditions, potential for large groundwater inflows at open fracture zones, and undetermined degree of hydraulic connection among fractures, landslide complexes, and geologic units.

4.2.2.1 SEM Tunnel

Access for the construction of the tunnel could be provided from both the North and South Portals. The South Portal approach structure would need to be largely completed to provide access from the south. A temporary or permanent bridge spanning the low area near the North Portal would need to be completed to provide access from the north.

Tunneling would ideally commence at the South Portal and proceed upgrade toward the North Portal to allow passive drainage of any groundwater entering at the excavation face in the tunnel.

Logistics for the equipment delivery and set-up need to be developed, as well as the plan to provide both temporary construction and permanent electrical service.

The volume of Alternative F excavation spoil is anticipated to be over one million cubic yards. Muck disposal areas in proximity to the site that can accommodate this volume of rock and soil would need to be identified.

An instrumentation program would have to be developed to monitor any impacts tunnel construction may have on the surrounding areas. This instrumentation should include but not be limited to piezometers to detect any changes in groundwater levels, extensometers to detect ground movements, and settlement monuments to detect surface settlement.

Because of the natural variability of the Broken Formation and the Melange, the SEM tunnel excavation face would often be in two or more different types of materials, such as hard sandstone and weathered argillite, each of which would behave differently during excavation. A flexible approach for SEM mining and support operations would be needed, with continual monitoring and adjustments as necessary to control face stability.

From the limited available data, sandstone of the Broken Formation spans a range of strengths and fracture spacings but includes rock which is relatively strong and potentially abrasive. Should the stronger and more abrasive rock occur over a longer length of the tunnel alignment than anticipated, tunnel progress could be significantly slowed, especially if the rock is less fractured than expected. Additional explorations and testing would help to constrain sandstone properties and their distribution along the tunnel alignment.

The relatively high quartz content of the sandstone indicates that a portion of the intact rock and the rock fragments to be encountered in excavations could be abrasive. If confirmed by additional petrographic analyses and abrasiveness testing, this could result in accelerated wear on excavation equipment as well as muck handling equipment, which would need to be considered in planning and scheduling.

From the currently available ATV/OTV data, the strike of the dominant joint set appears to be subparallel to the axis of the Alternative F tunnel and dipping less than 30 degrees west, a generally favorable condition. This orientation is classified as “Fair” by Bieniawski (1989) regardless of the direction of tunnel drive. Fractures in this joint set would daylight on the west-facing walls of the tunnel, possibly requiring additional support.

4.2.2.2 South Portal Approach with EDAS

Obstructions to excavation due to rock blocks within the Earthflow and Melange should be anticipated at surface-based excavations at the South Portal approach, including those for construction of the EDAS. While not evident from currently available boring information, large, intact blocks within these units have been observed in geologic mapping. If additional explorations in these areas suggest that such rock blocks could be present, pre-construction probe-hole drilling can reduce the likelihood of encountering unanticipated rock obstructions.

Retained excavation in argillite of the Melange at the South Portal and in the Broken Formation at the North Portal could be susceptible to slaking and deterioration upon exposure to air and water, based on current information (Caltrans, 2022a). IGM exposed in retained excavations could also be subject to failures on latent joint sets, as well as slaking or raveling. A protective sealant or shotcrete applied soon after excavation can reduce these risks.

Joint set orientations are not yet well defined in the project area, and the chaotic nature of the ground conditions may preclude the presence of site-specific joint sets. However, a recurrent joint set observed in ATV/OTV data strikes north-northwest and dips west. If present at the South and North Portals, this joint set would be subparallel to the long walls of proposed excavations and dipping out of and daylighting on the west-facing walls. If not supported, such joints could allow potentially unstable rock slabs or wedges to move into the excavation, especially from the east side.

4.2.2.3 North Portal and Bridge Approach

Because of the environmental sensitivity of this area, construction access requires careful consideration. Adjustments to construction methods may be required to minimize potential impacts to old-growth redwood trees.

5 RECOMMENDATIONS

5.1 Alternative X

5.1.1 Recommended Geotechnical Monitoring

On-going monitoring should be performed to further characterize the landslide complex as briefly described below.

Existing inclinometers should be continuously monitored to detect depth, direction, and magnitude of displacements and locations of failure zones. Measuring the basal failure displacements and incipient failure zones is critical to understanding and assessing landslide complex behavior. Additional inclinometer emplacements at proposed wall locations should be considered.

Existing and new VWPs should be continuously monitored to observe groundwater levels and their relationship to precipitation and landslide movements.

LiDAR surveys should be updated periodically (about every two to three years), including comparison with previous LiDAR surveys. These surveys can assist in assessing the relative landslide motion and the geometry of the landslide mass, as well as toe erosion rates.

Weather station and ocean swell data should be compiled and compared to inclinometer and groundwater data. This will help refine the understanding of the landslide driving mechanisms and their relationships and the impact of underground drainage to mitigating and/or slow the landslide movements to acceptable levels. This data will also be critical to developing the drain arrays, pipe sizing, and length extending into the landslides.

5.1.2 Recommended Geotechnical Investigations

For further development of the Alternative X design, future geotechnical investigations should include field mapping, additional borings, downhole geophysical testing, laboratory testing, and instrumentation. An objective of additional field work would be to better define basal failure surfaces so that drainage gallery tunnels can be properly located beneath them and so that the length and direction of the drainage pipes can be optimized for best drainage.

Field mapping to further characterize incipient landslides along Alternative X within the larger landslide complexes and to evaluate landslide movements relative to the structures' integrity and future maintenance should include geologic mapping along transects in various locations along the alignment including at proposed wall locations. Observing the location of surface features such as scarps, seeps and exposed intact rock blocks will refine the overall landslide modeling and refine the design of anchored soldier pile walls. Also, additional field mapping along the landslide toe along the beach should be performed to gain further insight into the landslide toe location and coastal erosion, with focus on the outfall tunnel area.

The boring locations and depths should be targeted to collect additional data for both the characterization of the overall landslide complexes as well as areas where new walls and underground drainage system structures are proposed. Borings for new walls should be spaced 100 to 200 feet along the proposed wall alignments and advanced at least to depths per the guidance of the Caltrans Geotechnical Manual for anchored soldier pile walls (2021e). Borings drilled should be instrumented with SI casing and VWPs for ongoing monitoring. In-situ geophysical testing should be performed at new borings, including borehole ATV/OTV logging and groundwater testing (pore water pressure tests, permeability tests, and pumping tests in open cased wells). Pressuremeter tests and P- and S-wave suspension logging should be performed in boreholes at locations where those data are needed for design.

Laboratory testing on samples collected from the borings should provide information to evaluate the stability and performance of the ERSs and include, at a minimum, moisture content/density tests, sieve analysis, No. 200 wash, corrosion, shear strength tests, and corrosion potential.

Collection of near-shore bathymetric topographic data may help characterize landslide geometries to evaluate effectiveness of the underground drainage system. Multiple surveys over time could provide useful data regarding changes in the bathymetric profile and relative rates of movement.

Additional pumping tests should be performed across the underground drainage area to collect groundwater flow data. Pumping test locations should be spaced such that at least four pumping tests are performed within each landslide complex. Each pumping test well should be drilled to a depth of at least 40 feet below the currently identified basal landslide failure plane from the nearest inclinometer reading. At least two sealed VWP boreholes with multiple, strategically placed transducers should be positioned within 15 feet of each pumping well. This data will be used to better understand fracture flow across the underground drainage system area and better define the size and character of the confined aquifer systems. Pumping test boreholes and/or adjacent VWP boreholes can be combined with gallery tunnel and shaft boreholes to reduce exploration costs.

Dye testing may provide gradient flow and flow rates of groundwater within the fractures as well as potential connectivity to seeps mapped at the toe of the slopes. This data will be used to further understand fracture connectivity and the flow rates will be critical to designing the drain pipes.

5.2 Alternative F

5.2.1 Recommended Geotechnical Monitoring

Existing and new SIs should be continuously monitored to detect depth, direction, and magnitude of displacements and locations of failure zones and earthflow movement. Inclinometers should be installed in all new borings in the vicinity of both portals and the earthflow area. Additional, inclinometers should be installed along the tunnel alignment.

Existing and new VWPs should be continuously monitored to observe and to baseline apparent groundwater levels and their relationship to precipitation and ground movements.

Periodic groundwater sampling and water quality analysis are recommended to establish baseline groundwater quality, its variation across the site, and its correlation with precipitation and ground movements. Depending on initial analysis results, an appropriate interval may be one sampling round per season. Analysis should include dissolved gases.

Monitoring of groundwater levels and water quality should continue for at least a one-year period prior to construction to established baseline conditions.

Weather station monitoring should also continue, increasing the number of stations if necessary.

LiDAR surveys should be updated periodically, including comparison with previous LiDAR surveys.

A complete geotechnical instrumentation and monitoring program would be needed to monitor site conditions during construction. A pre-construction baseline monitoring program of at least one year’s duration is recommended to establish baseline conditions against which construction conditions can be compared. Instrumentation would measure vertical and horizontal displacements at the ground surface and at depth, changes in groundwater levels and water quality, vibrations and noise levels, condition of existing structures, and other relevant factors.

5.2.2 Recommended Geotechnical Investigations

If Alternative F is selected, a comprehensive subsurface investigation is recommended to characterize subsurface conditions along the alignment for tunnel design and construction. An important objective of the additional geotechnical investigations would be confirmation of the location of the basal landslide failure surface.

Investigations could be carried out in phases, with each phase building on results of previous investigations and considering possible design revisions. Depending on the procurement method and timing, some investigations could be performed by the construction contractor. The scope of recommendations described below apply to investigations to be completed for final design.

Based on criteria of the U.S. Army Corps of Engineers (USACE, 1997), an appropriate boring spacing for SEM tunnels in this setting would be a maximum of about 500 feet, access permitting. These borings should ideally extend at least one to two tunnel diameters feet below the invert of the tunnel. Initially boring spacing could be wider, then reduced as additional explorations are performed during successive phases.

More detailed explorations are recommended at specific locations of proposed structures. In the South Portal area, borings are recommended along the secant pile wall locations for the cut-and-cover section and the EDAS. An expert-panel peer review is recommended to evaluate the proposed EDAS at the South Portal.

In the North Portal area, additional explorations are recommended to better define the limits of the NLCG landslide deposits and the basal failure surface. Additional structure-specific borings are also recommended for the OMC and Wilson Creek Tributary Bridge foundations.

In-situ testing should be performed at all new borings, including borehole ATV/OTV logging and permeability testing. Pressuremeter tests and P- and S-wave suspension logging should be performed in boreholes at locations where those data are needed for design.

Horizontal borings are recommended to better characterize highly variable or critical ground conditions, especially at the North Portal area where the location of the basal failure zone is uncertain. Even with boring spacing as close as 500 feet, it may not be otherwise possible to characterize the ground conditions with the level of detail needed to manage risk for design and construction.

Surface-based geophysical surveys, including resistivity and seismic refraction surveys, are recommended to supplement the proposed and existing exploratory borings in critical areas. Details would be developed as design and explorations are advanced.

Strategic borings should be instrumented with VWPs, removable transducers with data loggers and SIs and then regularly monitored.

Groundwater should be sampled and tested to determine a baseline condition for applicable water quality parameters, corrosivity, and dissolved gases.

A robust geotechnical laboratory testing program should be implemented to develop a statistically representative data set of properties for subsurface materials along the proposed Alternative F alignment. Laboratory test results would be used to develop baseline properties, to optimize selection of construction equipment and methods, and to refine cost and schedule estimates.

6 REFERENCES

Aalto, K.A. (1989). "Franciscan Complex Olistostrome at Crescent City, Northern California", in *Sedimentology* (1989) v. 36, p. 471-495.

Aalto, K.A. and Harper, G.D. (1982). *Geology of the Coast Ranges in the Klamath and Ship Mountain quadrangles, Del Norte County, California*, California Division of Mines and Geology Open-File Map QFR 82-16-SF, map scale 1:62,500.

Anderson, D.G, Martin, G.R, Lam, I., and Wang, J.N. (2008). *Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments*, NCHRP Report 611, Transportation Research Board.

Atwater, B.F., Musumi-Rokkaku, S., Satake, K., Tsuji, Y., Ueda, K., and Yamaguchi, D.K. (2005). *The Orphan Tsunami of 1700 – Japanese Clues to a Parent Earthquake in North America (Second Edition)*. University of Washington Press and U.S. Geological Survey Professional Paper 1707. https://doi.org/10.3133/pp1707.

Barton, N. (2002). "Some New Q-Value Correlations to Assist in Site Characterization and Tunnel Design," *International Journal of Rock Mechanics & Mining Sciences*, Elsevier Science, Ltd., vol. 39, pp. 185-216.

Belitz, K., Dubrovsky, N.M., Burow, K.R., Jurgens, B., and Johnson, T. (2003). *Framework for a Groundwater Quality Monitoring and Assessment Program for California*: U.S. Geological Survey Water-Resources Investigations Report 03-4166, 78 p.

BGC Engineering USA Inc. (2018). *Last Chance Grade Expert-Based Risk Assessment, Final,* prepared for California Department of Transportation, Project No.: 1776001. June 14, 2018, 61 p.

Bieniawski, Z. T. (1989). *Engineering Rock Mass Classifications: a Complete Manual for Engineers in Mining, Civil, and Petroleum Engineering*, New York: John Wiley, 251 p.

Bieniawski, Z. T. (1992). *Design Methodology in Rock Engineering*, London: CRC Press, 210 p.

Bryant, W.A., compiler (2017). "Fault Number 147, Lost Man Fault," *in Quaternary Fault and Fold Database of the United States*: U.S. Geological Survey website. https://earthquake.usgs.gov/cfusion/qfault/show_report_AB_archive.cfm?fault_id=147&section_id=, accessed 06/09/2021.

California Building Standards Commission (CBSC) (2019). *2019 California Building Standards Code,* Title 24, California Code of Regulations (Title 24), July 1, 2019.

California Department of Water Resources (DWR) (2021). *California's Groundwater Update 2020*, Bulletin 118.

California Geological Survey (CGS) (2007). *Fault-Rupture Hazard Zones in California*, Special Publication 42, Interim Revision.

California Office of Environmental Health Hazard Assessment (2023). https://oehha.ca.gov/climate-change/epic-2022/impacts-physical-systems/sea-level-rise#.Caltrans (2010). *Soil and Rock Logging, Classification, and Presentation Manual*, 2010 Edition. State of California Department of Transportation, Division of Engineering Services, Geotechnical Services.

Caltrans (2012). *Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations*, November.

Caltrans (2013). Memo to Designers 20-10, Fault Rupture, January.

Caltrans (2014). Geotechnical Manual – Embankments, December.

Caltrans (2015). *Last Chance Grade Engineered Feasibility Study*, Project Location On Route 101 in Del Norte County, 10 miles south of Crescent City from PM 12 - 15.5, EA 01-987101, June 25, 2015.

Caltrans (2016). Monitoring Results Report: Fiscal Year 2015–2016. CTSW-RT-16-312.01.02.

Caltrans (2017). Geotechnical Manual – Fault Rupture, January.

Caltrans (2018). *Summary of Phase 1 Geotechnical Investigation*. California Department of Transportation, Division of Engineering Services, Geotechnical Services. File: 01-DN 101-PM 12.0 to 15.5, EA 01-0F280, EFIS: 0115000099. May 17, 2018.

Caltrans (2019a). Seismic Design Criteria, Version 2.0, April.

Caltrans (2019b). October 2019 Interim Revisions to Seismic Design Criteria Version 2.0, October.

Caltrans (2020a). Geotechnical Manual - Design Acceleration Response Spectrum, January.

Caltrans (2020b). Geotechnical Manual – Landslides, January.

Caltrans (2020c). Geotechnical Manual – Liquefaction-Induced Lateral Spreading, January.

Caltrans (2020d). Geotechnical Manual – Liquefaction Evaluation, January.

Caltrans (2020e). Caltrans Water Quality Planning Tool, url: http://svctenvims.dot.ca.gov/wqpt/wqpt.aspx, accessed: November 2021.

Caltrans (2021a). Last Chance Grade Permanent Restoration Project, *Hydrology and Surface Water Technical Memorandum*, SUB#014_R01, EA# 01-0F280, Project EFIS# 0115000099, Del Norte County, U.S. 101, PM 12.0/15.5, dated June 24, 2021, 24 p.

Caltrans (2021b). Caltrans Geotechnical Manual, Design Acceleration Response Spectrum, Attachment 2.

Caltrans (2021c). ARS Online Web Tool V3.0.2, https://arsonline.dot.ca.gov/.

Caltrans (2022a). Last Chance Grade Permanent Restoration Project, *Preliminary Geotechnical Data Report – Final*, SUB#032, EA# 01-0F280, Project EFIS# 0115000099, Del Norte County, U.S. 101, PM 12.0/15.5, dated July 2022, 64 p., 13 appendices.

Caltrans (2022b). Last Chance Grade Permanent Restoration Project, *Hydrogeology Report – Draft,* SUB#076, EA# 01-0F280, Project EFIS# 0115000099, Del Norte County, U.S. 101, PM 12.7/16.5, dated October 2022, 81 p., 1 appendix.

Caltrans (2022c). Last Chance Grade Permanent Restoration Project, *Aquifer Pumping Test Evaluations, Technical Memorandum,* SUB#040, EA# 01-0F280, Project EFIS# 0115000099, Del

Norte County, U.S. 101, PM 12.7/16.5, dated December 2022, 24 p., 5 appendices.

Caltrans (2023). Last Chance Grade Permanent Restoration Project, *Draft Project Report*, SUB#095, EA# 01-0F280, Project EFIS# 0115000099, Del Norte County, U.S. 101, PM 12.7/16.5, dated October 2023, 159 p., 13 attachments.

Chaytor, J.D., Goldfinger, C., Dziak, R.P., and Fox, C.G. (2004). “Active Deformation of the Gorda Plate: Constraining Deformation Models with New Geophysical Data”, in *Geology*, v. 32, no. 4) p. 353.

Clarke, S.H., Jr. (1992). “Geology of the Eel River Basin and Adjacent Region: Implications for Late Cenozoic Tectonics of the Southern Cascadia Subduction Zone and Mendocino Triple Junction.” *AAPG Bulletin*, v. 76, no. 2, p. 199-224.

Clarke, S.H., Jr. and Carver, G.A. (1992). “Late Holocene Tectonics and Paleoseismicity of the Southern Cascadia Subduction Zone, Northwestern California”. *Science*, v. 255, p. 188-192.

Deere, D.U. and D.W. Deere (1988). “The Rock Quality Designation (RQD) Index in Practice,” *Rock Classification Systems for Engineering Purposes*, ASTM STP 984, L. Kirkaldie, ed., Philadelphia, pp. 91-101.

Delattre, M. and Rosinski, A. (2012). *Preliminary Geologic Map of Onshore Portions of the Crescent City and Orick 30’ X 60’ Quadrangles, California,* map and pamphlet, California Department of Conservation California Geological Survey, map scale 1:100,000.

Dengler, L.A. (2008). “The 1906 Earthquake on California’s North Coast.” *Bulletin of the Seismological Society of America*, v. 98, no. 2, 918-930.

Freymueller, J.T., Murray, M.H., Segall, P. and Castillo, D. (1999). “Kinematics of the Pacific-North America Plate Boundary Zone, Northern California.” *Journal of Geophysical Research: Solid Earth*, v. 104, no. B4, p. 7419-7441.

Furlong, K.P. and Schwartz, S.Y. (2004). “Influence of the Mendocino Triple Junction on the Tectonics of Coastal California.” *Annual Review of Earth & Planetary Sciences*, v. 32, no. 1, p. 403-433.

Hart, E.W., compiler (1999). “Fault Number 11, Grogan Fault,” in *Quaternary Fault and Fold Database of the United States*, U.S. Geological Survey website. https://earthquake.usgs.gov/cfusion/qfault/show_report_AB_archive.cfm?fault_id=11&section_id=, accessed 06/09/2021.

Heaton, T.H. and Hartzell, S.H. (1987). “Earthquake Hazards on the Cascadia Subduction Zone.” *Science*, v. 236, no. 4798, p. 162-168.

Hemphill-Haley, M.A., Hemphill-Haley, E. and Wunderlich, W. (2020). “Overview of Geological Hazards.” *California North Coast Offshore Wind Studies*, M. Severy, Z. Alva, G. Chapman, M. Cheli, T. Garcia, C. Ortega, N. Salas, A. Younes, J. Zoellick, and A. Jacobson (Eds.), Humboldt, CA: Schatz Energy Research Center. 126 p. https://schatzcenter.org/pubs/2020-OSW-R16.pdf.

Kelsey, H.M., and Carver, G.C. (1988). “Late Neogene and Quaternary tectonics associated with northward growth of the San Andreas Transform Fault, northern California. *Journal of Geophysical Research: Solid Earth*, v. 93, no. B5, p. 4797-4819.

Marinos, V., Marinos, P., and Hoek, E., 2005. "The Geological Strength Index- Applications and Limitations," *Bul Eng Geol Env* 64:55-65.

Mathany, T.M., Dawson, B.J., Shelton, J.L. and Belitz, K. 2011. *Groundwater-Quality Data in the Northern Coast Ranges Study Unit, 2009: Results from the California Groundwater Ambient Monitoring and Assessment (GAMA) Program*: U.S. Geological Survey Data-Series 609, 92 p.

Mathany, T.M. and Belitz, K. 2015. *Status and Understanding of Groundwater Quality in the Northern Coast Ranges Study Unit, 2009—California GAMA Priority Basin Project*: U.S. Geological Survey Scientific Investigations Report 2014–5215, 86 p.

McLaughlin, R.J., Ellen, S., Blake Jr. M., Jayko, A.S., Irwin, W., Aalto, K. Carver, G., and Clarke, Jr. S. (2000). *Geology of the Cape Mendocino, Eureka, Garberville, and Southwestern Part of the Hayfork 30 x 60 Minute Quadrangles and Adjacent Offshore Area, Northern California*: U.S. Geological Survey Miscellaneous Field Studies MF-2336.

Nelson, A.R., DuRoss, C.B., Witter, R.C., Kelsey, H.M., Engelhart, S.E., Mahan, S.A., Gray, H.J., Hawkes, A.D., Horton, B.P., and Padgett, J.S. (2021). "A Maximum Rupture Model for the Central and Southern Cascadia Subduction Zone- Reassessing Ages for Coastal Evidence of Megathrust Earthquakes and Tsunamis", *Quaternary Science Reviews*, v. 261 (2021), article no. 106922.

North Coast Regional Water Quality Control Board (RWQCB) (2016). "Waste Discharge Requirements for Discharges of Highly Treated Groundwater to Surface Waters Following Extraction and Treatment of Groundwater Polluted with Petroleum Hydrocarbons and Volatile Organic Compounds." Order No. R1-2016-0034, General NPDES No. CAG911001. 93 p.

North Coast Regional Water Quality Control Board (RWQCB) (2018). "Water Quality Control Plan for the North Coast Region." June 2018 Edition. 203 p.

PNSN (2020). Cascadia Subduction Zone. Pacific Northwest Seismic Network, https://pnsn.org/outreach/earthquakesources.

Ristau, D. (1979). Geologic Map, Klamath 15' Quadrangle: California Department of termporry, Title II Geologic Data Compilation Project, unpublished, map scale 1:62,500.

Rocscience, Inc. (2021a). *DIPS Software Program for Graphical and Statistical Analysis of Orientation Data,* version 8.011, Rocscience, Toronto, Ontario, Canada

Rocscience Inc. (2021b). Slide2 Modeler, 2D Limit Equilibrium Analysis for Slopes, Build: 9.018 64bit, Build date: June 28, 2021, 06:00:53.

Rollins, J.C., and Stein, R.S. (2010). "Coulomb Stress Interactions Among M>5.9 Earthquakes in the Gorda Deformation Zone and on the Mendocino Fault Zone, Cascadia Subduction Zone, and Northern San Andreas Fault", *Journal of Geophysical Research*, v. 115, B12306, p. 1-19.

State Water Resources Control Board (SWRCB) (2019). "California Ocean Plan, Water Quality Control Plan, Ocean Waters of California." Established 1972, Revised 2019. 117 p.

Tokimatsu, K. and Seed, H. B. (1987). "Evaluation of Settlements in Sand Due to Earthquake Shaking," *Journal of Geotechnical Engineering*, ASCE, Vol. 113, No. 8, pp.861-878.

United States Army Corps of Engineers (USACE), 1997. *Engineering and Design, Tunnels and Shafts in Rock,* CECW-EG Engineer Manual EM 1110-2-2901, Washington, D.C.: Department of the Army, U.S. Army Corps of Engineers.

United States Department of Agriculture (USDA), Natural Resources Conservation Service, United States Department of Agriculture. Redwood National and State Parks (CA796), url: http://websoilsurvey.sc.egov.usda.gov/, accessed November 1, 2021.

United States Department of Agriculture (USDA), National Resources Conservation Service, United States Department of Agriculture. Humboldt and Del Norte Area, California, (CA605), url: http://websoilsurvey.sc.egov.usda.gov/, accessed November 1, 2021.

U.S. Department of the Interior, Bureau of Reclamation (USBR) (1976). *Preconstruction Geology Pacheco Tunnel Reach 2, Central Valley Project, San Felipe Division,* USBR, Mid-Pacific Region, Sacramento, California.

U.S. Geological Survey (USGS) (1997a). *Childs Hill Quadrangle, California-Del Norte Co.*, 7.5-Minute Series (Topographic), scale 1:24000.

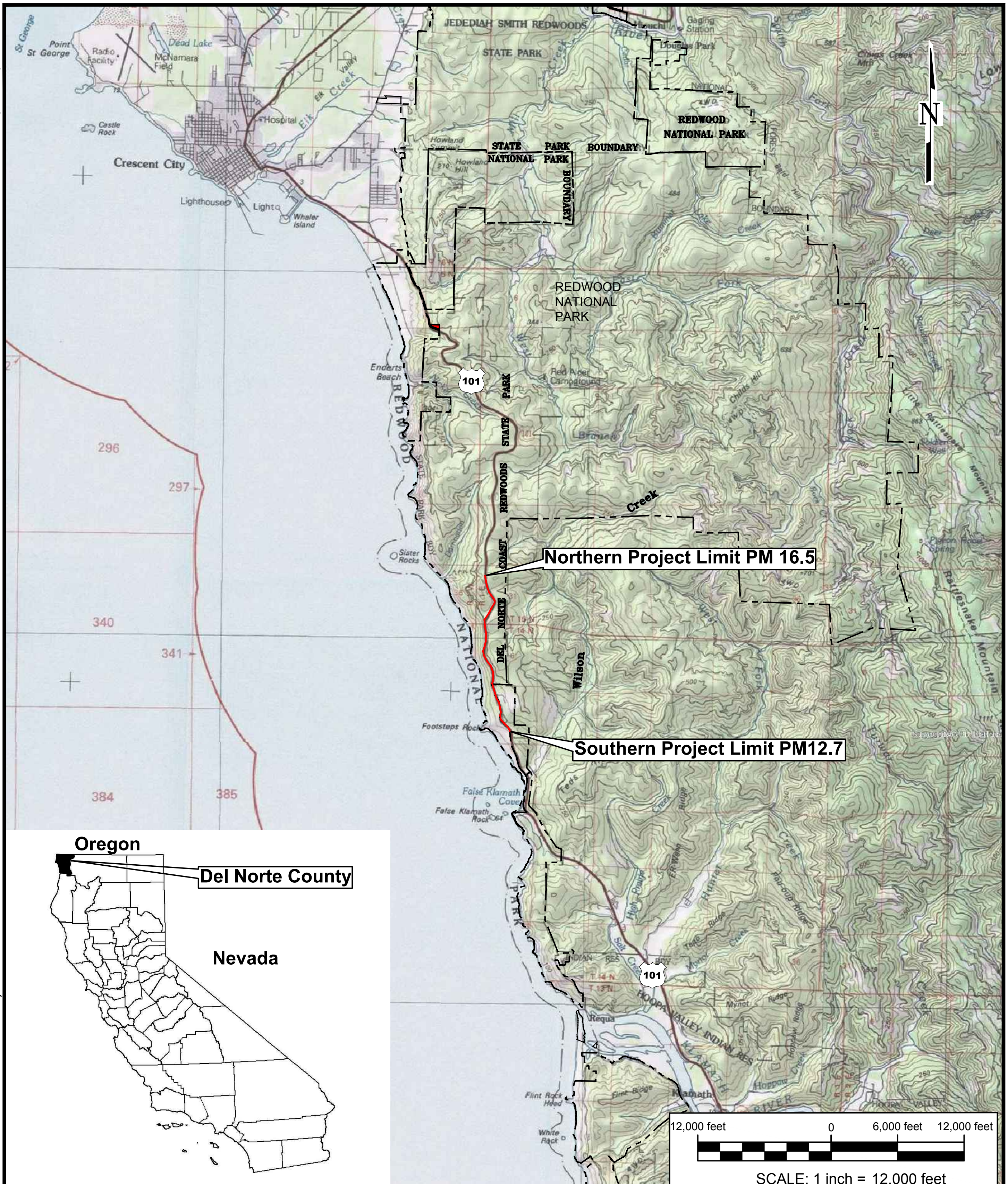
U.S. Geological Survey (USGS) (1997b). *Requa Quadrangle, California-Del Norte Co.,* 7.5-Minute Series (Topographic), scale 1:24000.

United States Geological Survey (USGS) (2013). *The Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) – The Time Independent Model*, USGS Open File Report 2013-1165, CGS Special Report 228, Southern California Earthquake Center Publication 1792.

Wills, C.J. (2000). *Landslides in the Highway 101 Corridor Between Wilson Creek and Crescent City, Del Norte County, California*. California Geological Survey Special Report 184, 26 p, 2 plates.

Youd, T.L., and Idriss, I.M. (2001). Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 127, No. 10, October.

PLATES



LAST CHANCE GRADE PERMANENT RESTORATION PROJECT
PRELIMINARY GEOTECHNICAL REPORT - FINAL

SITE LOCATION PLAN

PLATE
1

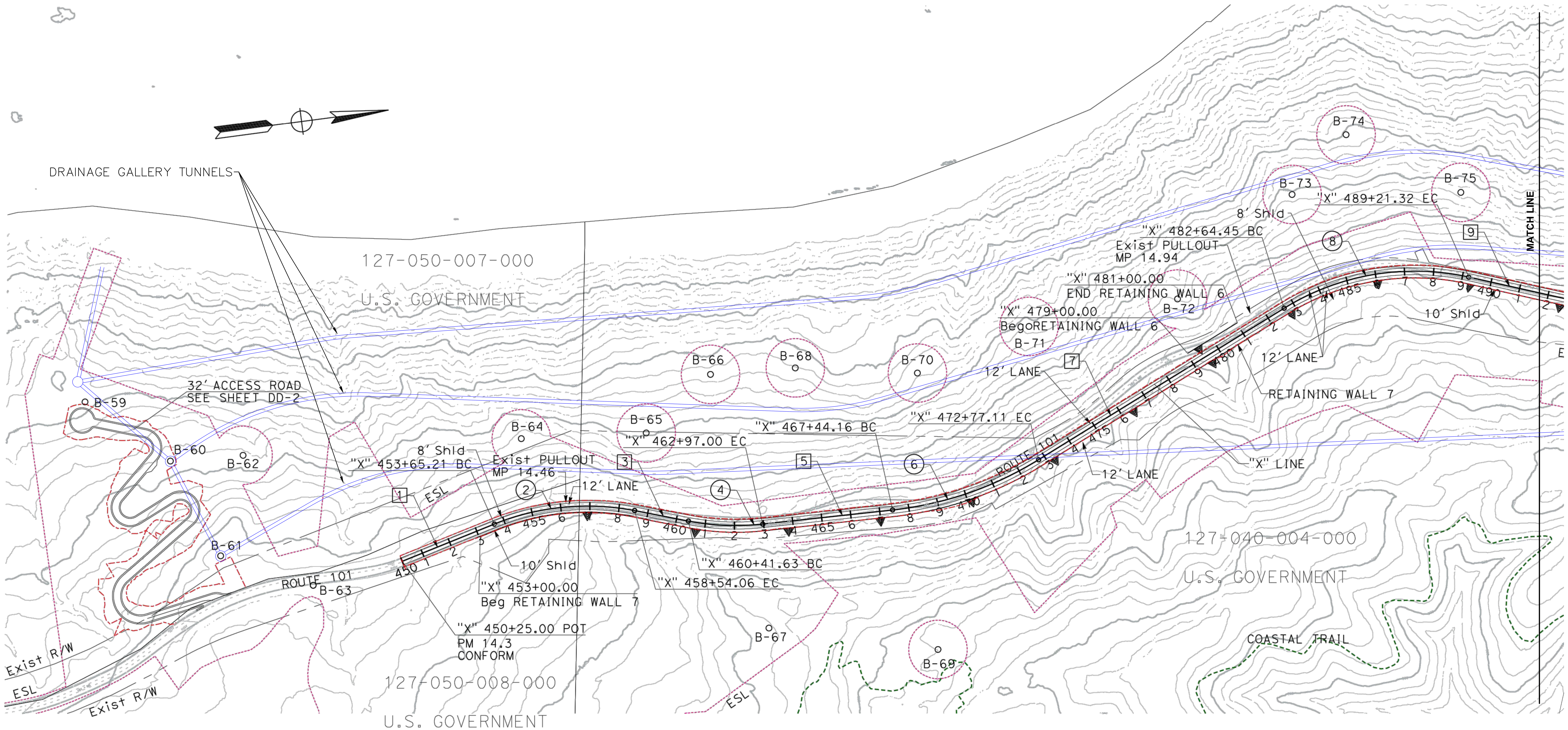
PLOTTED: 08 Nov 2023, 11:03am, JSala

LAYOUT: 2a

CAD FILE: W:\2024\20208000.001A - Last Chance Grade\

LEGEND:

- CUT/FILL LINE
- COASTAL TRAIL
- ENVIRONMENTAL STUDY LIMITS (ESL)
- Exist R/W
- STRUCTURE
- RETAINING WALL
- B-XX ○ BORING LOCATION



Notes:
1. Data source: Layout Alternative X, L-1 Caltrans (2023).
2. Design and alignments shown for Alternatives X are current as of October 2023.

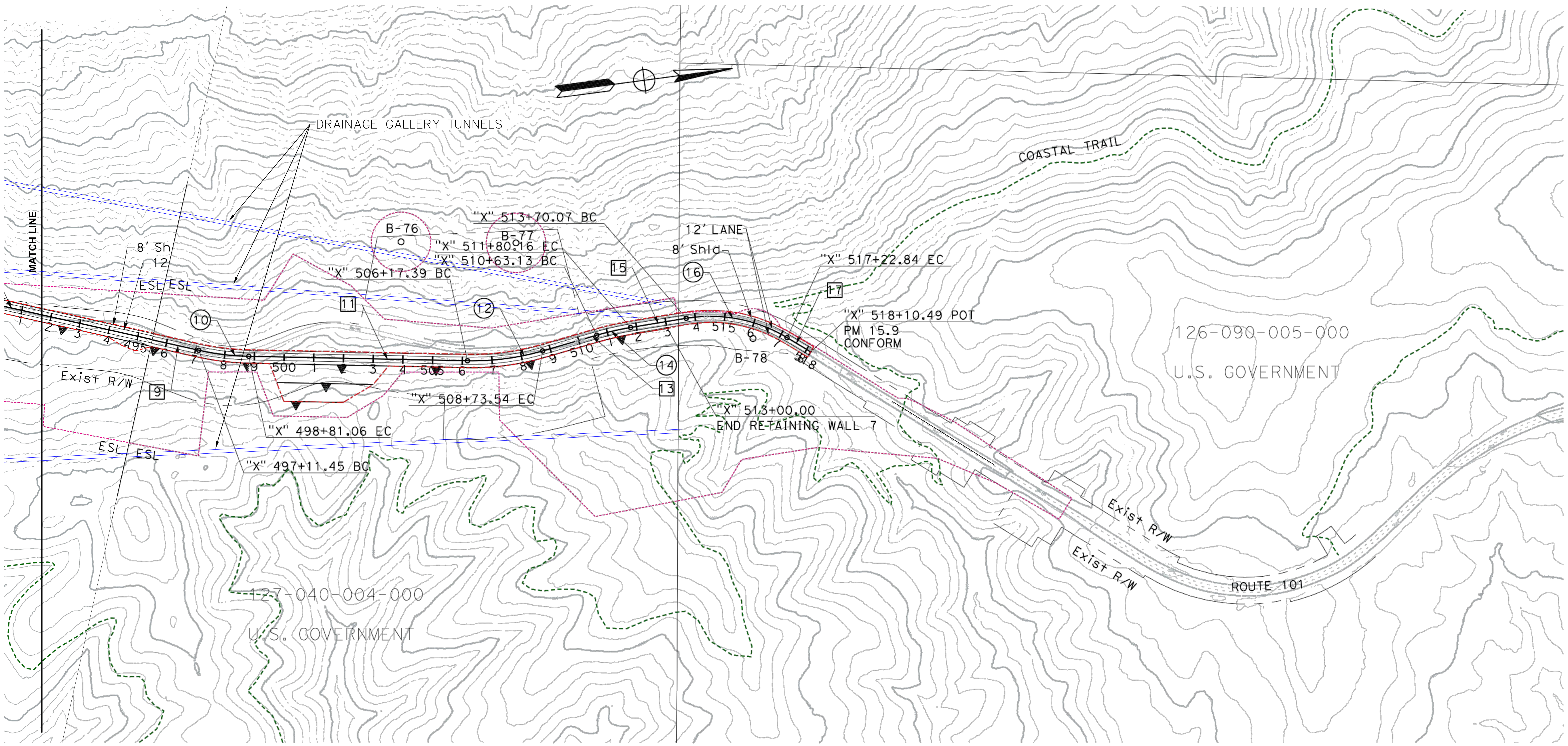
200 0 100 200 400
APPROXIMATE SCALE: 1 inch = 200 feet

LAST CHANCE GRADE PERMANENT RESTORATION PROJECT PRELIMINARY GEOTECHNICAL REPORT - FINAL	**ALTERNATIVE X PLAN VIEW**	PLATE **2a**

PLOTTED: 08 Nov 2023, 11:05am, JSala

LEGEND:

- - - - (red)	CUT/FILL LINE
- - - - (green)	COASTAL TRAIL
······ (magenta)	ENVIRONMENTAL STUDY LIMITS (ESL)
—— — —	Exist R/W
—— (blue)	STRUCTURE
—▲—▲—	RETAINING WALL
B-XX o	BORING LOCATION



Notes:
1. Data source: Layout Alternative X, L-1 Caltrans (2023).
2. Design and alignments shown for Alternatives X are current as of October 2023.

200 0 100 200 400
APPROXIMATE SCALE: 1 inch = 200 feet

CAD FILE: W:\2024\20208000.001A - Last Chance Grade\ LAYOUT: 2b

LAST CHANCE GRADE PERMANENT RESTORATION PROJECT
PRELIMINARY GEOTECHNICAL REPORT - FINAL

ALTERNATIVE X PLAN VIEW

PLATE
2b

PLOTTED: 08 Nov 2023, 11:54am, JSala

LEGEND:

(brown dashed line)	CUT/FILL LINE
(green dotted line)	COASTAL TRAIL
(magenta line)	ENVIRONMENTAL STUDY LIMITS (ESL)
(grey dashed line)	Exist R/W
(blue line)	STRUCTURE
(black line with ticks)	RETAINING WALL
(purple line)	DRAINAGE STRUCTURE
B-XX o	BORING LOCATION

Notes:
1. Data source: Layout Alternative F, L-2 Caltrans (2023).
2. Design and alignments shown for Alternatives F are current as of October 2023.

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APPROXIMATE SCALE: 1 inch = 200 feet

LAST CHANCE GRADE PERMANENT RESTORATION PROJECT PRELIMINARY GEOTECHNICAL REPORT - FINAL	**ALTERNATIVE F PLAN VIEW**	PLATE **3a**

CAD FILE: W:\2024\20208000.001A - Last Chance Grade\ LAYOUT: 3a

PLOTTED: 08 Nov 2023, 11:55am, JSala

LAYOUT: 3b

CAD FILE: W:\2024\20208000.001A - Last Chance Grade\

LEGEND:

- CUT/FILL LINE
- COASTAL TRAIL
- ENVIRONMENTAL STUDY LIMITS (ESL)
- Exist R/W
- STRUCTURE
- RETAINING WALL
- DRAINAGE STRUCTURE
- B-XX o BORING LOCATION

Notes:
1. Data source: Layout Alternative F, L-2 Caltrans (2023).
2. Design and alignments shown for Alternatives F are current as of October 2023.

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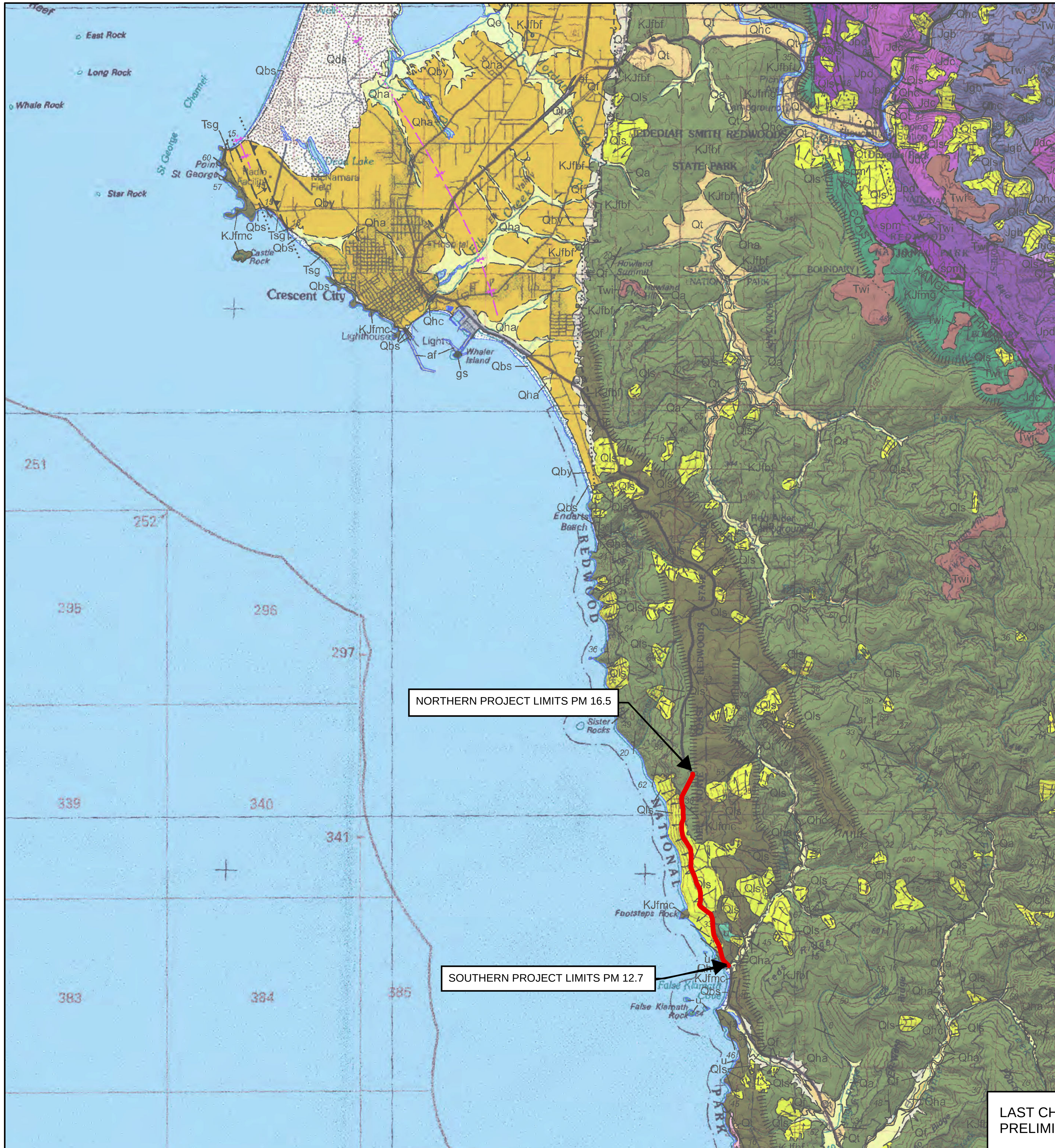
APPROXIMATE SCALE: 1 inch = 200 feet

LAST CHANCE GRADE PERMANENT RESTORATION PROJECT
PRELIMINARY GEOTECHNICAL REPORT - FINAL

ALTERNATIVE F PLAN VIEW

PLATE

3b



EXPLANATION

Symbol	Description
af	Artificial fill (historical)
Qhc	Stream channel deposits (latest Holocene)
Qbs	Beach deposits (latest Holocene)
Qha	Young alluvial deposits, undifferentiated (Holocene)
Qhf	Young alluvial fan deposits (Holocene)
Qht	Young stream terrace deposits (Holocene)
Qds	Dune sand (Holocene)
Qe	Estuarine deposits (Holocene)
Qls	Landslide deposits (historical to Pleistocene)
Qa	Alluvial deposits, undifferentiated (Holocene to latest Pleistocene)
Qf	Alluvial fan deposits (Holocene to Pleistocene)
Qt	Stream terrace deposits (early Holocene to Pleistocene)
Qby	Battery Formation (late Pleistocene)
Qsc	Terrace gravels of Surpur Creek (Pleistocene)
Qu	Undifferentiated marine and nonmarine overlap deposits (Pleistocene to late Pliocene?)
QTpc	Prairie Creek Formation (early Pleistocene to late Pliocene)
Twi	Wimer Formation (late Miocene)
Tsg	St. George Formation (late Miocene)

Franciscan Complex - Central Belt

Symbol	Description
KJfm	Mélange of the Central Belt (Late Cretaceous to Late Jurassic)
gs	Greenstone block within mélange

Franciscan Complex - Eastern Belt

Symbol	Description
KJfbf	Broken formation (Early Cretaceous to Middle Jurassic)
KJfmc	Mélange unit of Crescent City area (Early Cretaceous to Middle Jurassic)
	Blocks within mélange:
gs	Greenstone
ch	Chert
mg	Metagraywacke
u	Undifferentiated
KJfmg	Metagraywacke (Cretaceous to Jurassic)

Klamath Mountains Province - Western Jurassic Belt

Josephine Ophiolite of Harper (1980)

Symbol	Description
Jdc	Dike Complex (Late Jurassic)
Jgb	Gabbro (Late Jurassic)
Jpd	Peridotite (Late Jurassic)
Juc	Cumulate ultramafic rocks (Late Jurassic)
sp	Sheared serpentinite (Jurassic)
spm	Serpentinite matrix mélange (Jurassic)
	Blocks within mélange:
gs	Greenstone
gr	Granitic rocks
ms	Metasedimentary rocks

SYMBOL EXPLANATION

Contact between map units - Hashed where gradational; solid where accurately located, dashed where approximately located; short dash where inferred; dotted where concealed; queried where uncertain.

Fault - Solid where accurately located; dashed where approximately located; short dash where inferred; dotted where concealed; queried where uncertain; bar and ball on downthrown side. U = upthrown block, D = downthrown block.

Thrust fault - Sawteeth on upper plate; solid where accurately located; dashed where approximately located; short dash where inferred; dotted where concealed; queried where existance uncertain

Anticline - Solid where accurately located, dashed where approximately located; dotted where concealed.

Syncline - Solid where accurately located, dashed where approximately located; dotted where concealed.

Strike and dip of bedding:

- Inclined
- Vertical
- Overturned
- Crumpled
- Approximate

Strike and dip of foliation:

- Inclined
- Vertical

Strike and dip of joints or fractures

Strike and dip if igneous layering

Strike and dip of dike walls

Landslide - arrows indicate principal direction of movement.

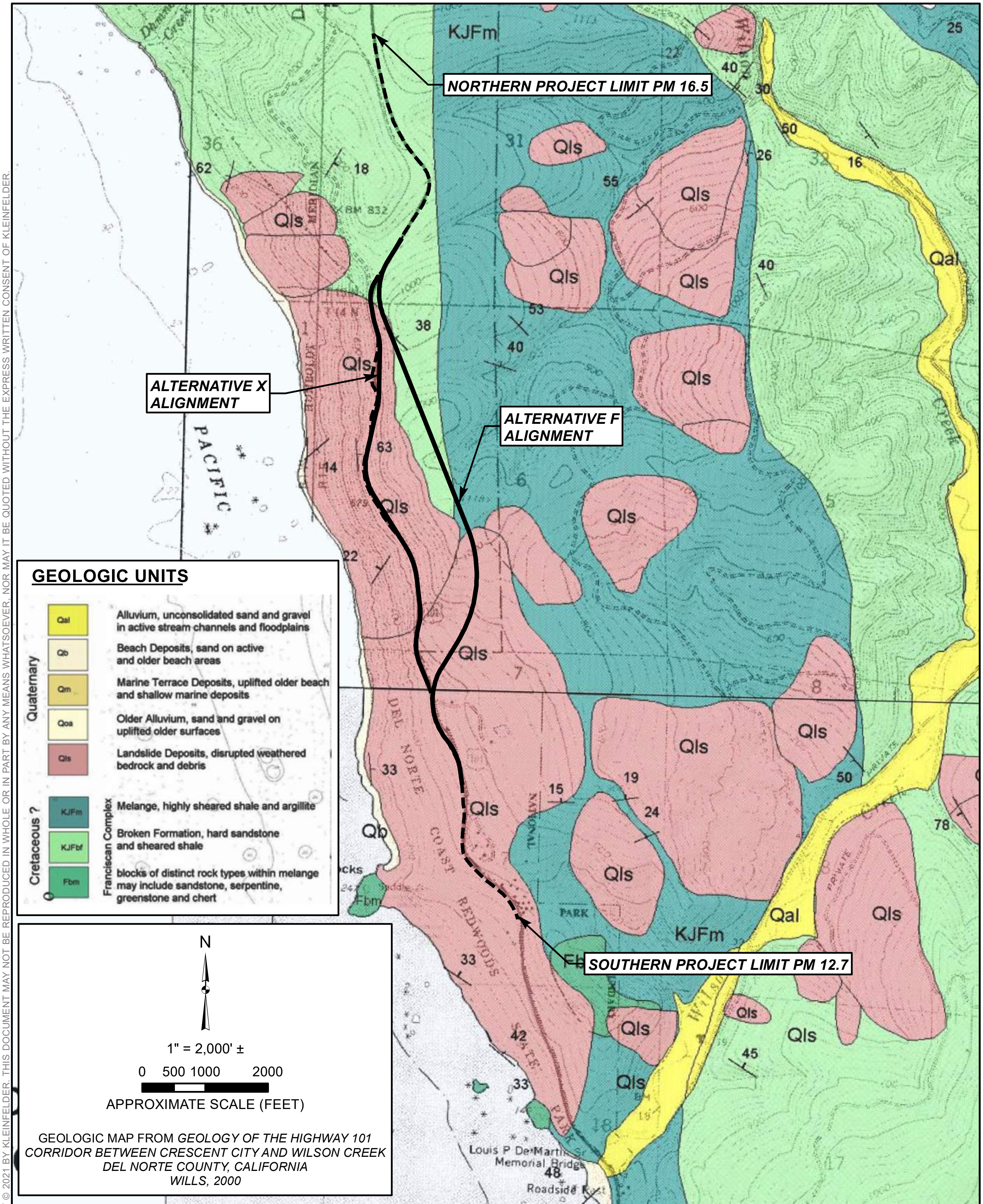
1"=9,000'±

0 9,000

FEET

GEOLOGIC MAP FROM PRELIMINARY GEOLOGIC MAP OF ONSHORE PORTIONS OF THE CRESCENT CITY AND ORICK 30' X 60' QUADRANGLES, CALIFORNIA, DELATTRE AND ROSINSKI, 2012

LAST CHANCE GRADE PERMANENT RESTORATION PROJECT PRELIMINARY GEOTECHNICAL REPORT - FINAL	PUBLISHED GEOLOGIC MAP DELATTRE and ROSINSKI (2012)	PLATE 4



LAST CHANCE GRADE PERMANENT RESTORATION PROJECT PRELIMINARY GEOTECHNICAL REPORT - FINAL	PUBLISHED GEOLOGIC MAP, WILLS (2000)	PLATE 5

LANDSLIDE IDENTIFICATION CHART

1 2 3 4

STATE OF ACTIVITY

1 = Active or Recently Active

(areas of unstable ground with relatively recent/"fresh" geomorphic features such as ground cracks, hummocky topography, exposed soils, abrupt gradient breaks and/or disrupted vegetation, typically recent to 50 years old)

2 = Dormant

(areas of quasi-stable ground, with eroded and subdued geomorphic features, no exposed soils, somewhat re-vegetated but typically with different type or density, typically >50 to several hundreds of years old)

3 = Ancient

(areas of relatively stable ground, typically characterized by large, broad and deep landslides with highly eroded and subdued geomorphic features, re-vegetated with similar type and density, typically several hundreds to several thousands of years old)

CERTAINTY OF IDENTIFICATION

1 = Definite
2 = Probable
3 = Questionable

DOMINANT TYPE OF MOVEMENT

1 = Slump-Flow Complex
2 = Debris Slide
3 = Debris Flow
4 = Earth Flow
5 = Slump*
6 = Translational*
7 = Rockfall/Topple
8 = Wedge Slide

THICKNESS OF DEPOSIT

1 = Less Than 5 Feet
2 = 5 to 15 Feet
3 = 15 to 50 Feet
4 = Greater Than 50 Feet

* Classification includes either soil-like or rock landslides. Classifications modified after Varnes (1978), v.2-2012

GEOLOGIC UNITS

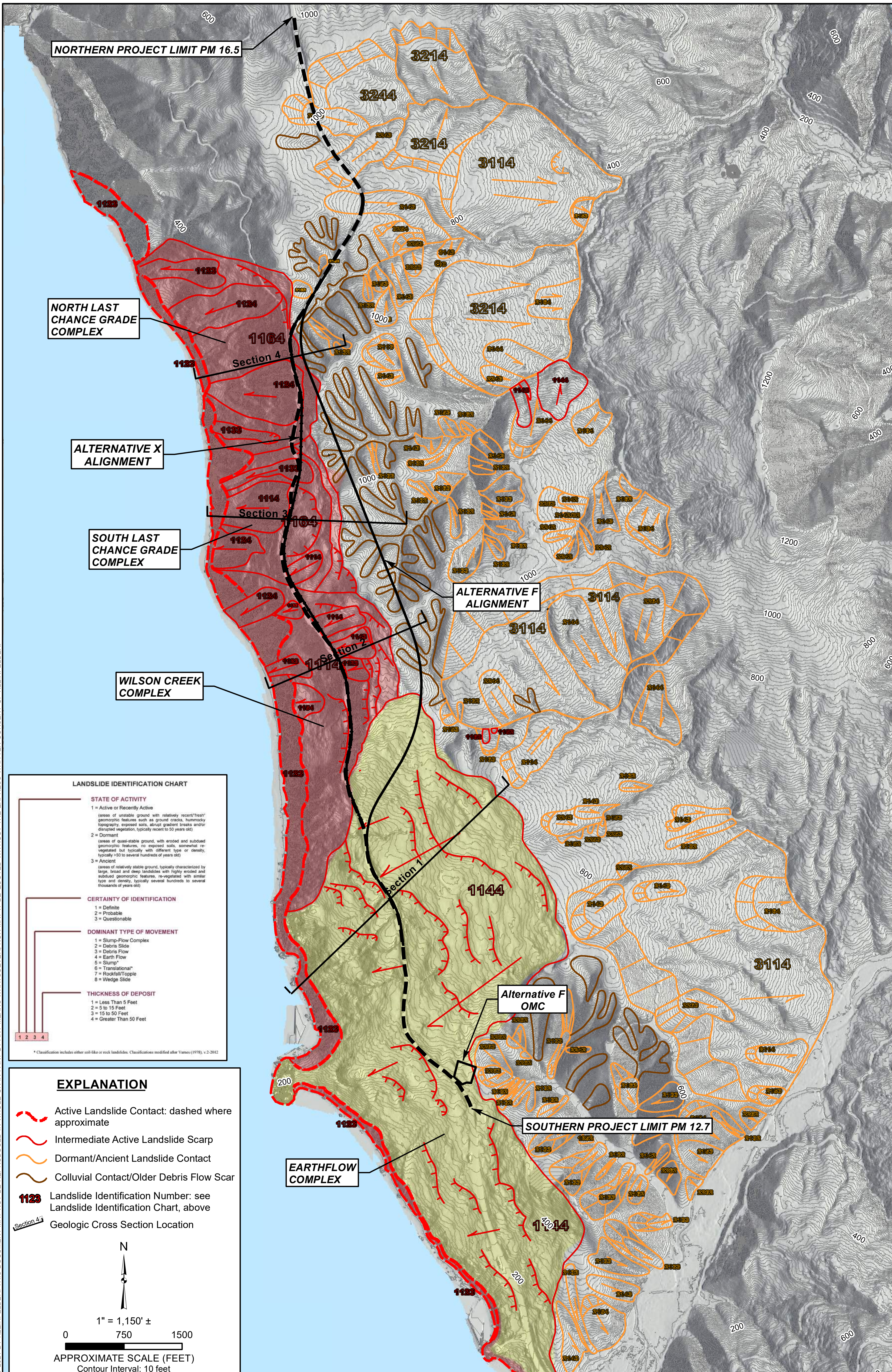
Symbol	Geologic Unit	Description
Qal	Alluvium	Sand and sandy gravel with some fine-grained soil
Qc	Colluvium	Loose, heterogeneous mass of soil and/or rock fragments transported and deposited downslope by sheet flow or slow, continuous creep
Qlsd-m	Earthflow Landslide Deposits, Derived from Melange	Landslide deposits consisting of a mixture of fine-grained soils, deeply weathered rock, and scattered sandstone clasts shich have been transported as a sliding mass with many internal slip surfaces
Qlsd-bf	Rock/Debris Landslide Deposits Derived from Broken Formation	Landslide deposits consisting of blocks of sandstone and argillite rock and/or debris which have been transported by sliding falling
KJFm	Franciscan Complex Melange	Dark gray, pervasively sheared, soil-like argillite with scattered blocks of intact sandstone
KJFbf	Franciscan Complex Broken Formation	Blocks of gray, hard, massive to very thickly bedded sandstone with interbedded argillite separated by weak, sheared zones

CAD FILE: K:\2020 PROJECTS\2020800.001A LAST CHANCE GRADE\Cross Sections and Alignments\ LAYOUT: Layout1

LAST CHANCE GRADE PERMANENT RESTORATION PROJECT
PRELIMINARY GEOTECHNICAL REPORT - FINAL

LANDSLIDE CLASSIFICATION CHART AND GEOLOGIC UNITS DESCRIPTIONS

PLATE
6

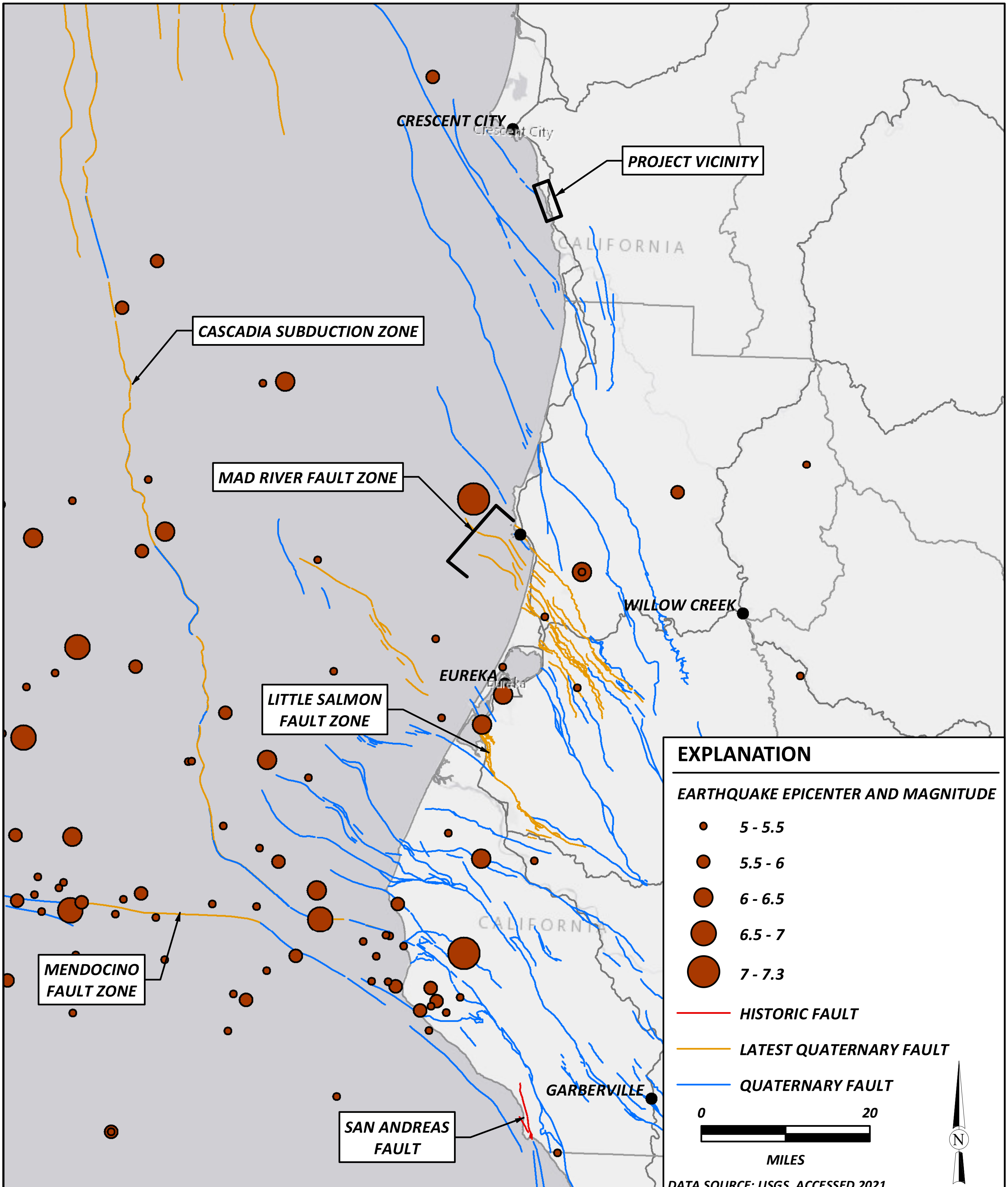


LAST CHANCE GRADE PERMANENT RESTORATION PROJECT

PRELIMINARY GEOTECHNICAL REPORT - FINAL

PROJECT LANDSLIDE MAP

PLATE 7



LAST CHANCE GRADE PERMANENT RESTORATION PROJECT PRELIMINARY GEOTECHNICAL REPORT - FINAL	REGIONAL SEISMICITY AND FAULT MAP	PLATE 8

PLOTTED: 07 Nov 2023, 11:08am, JSala
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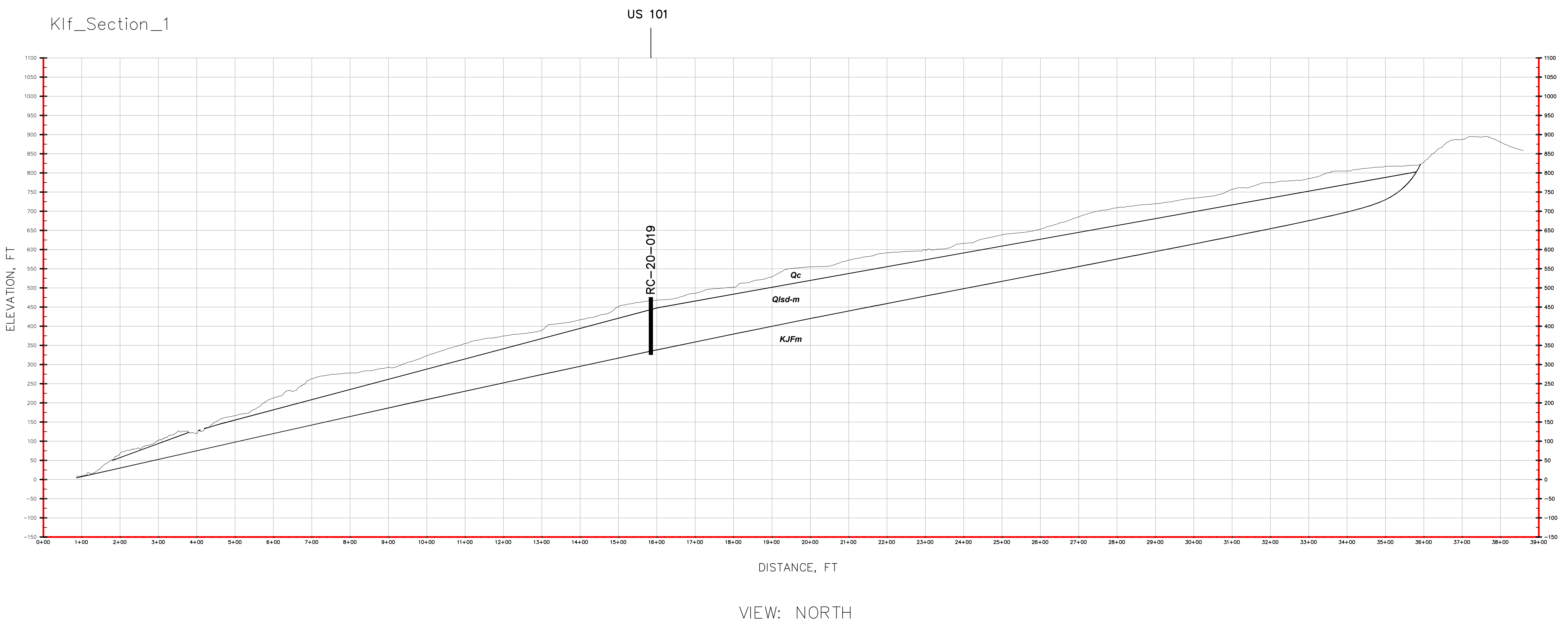
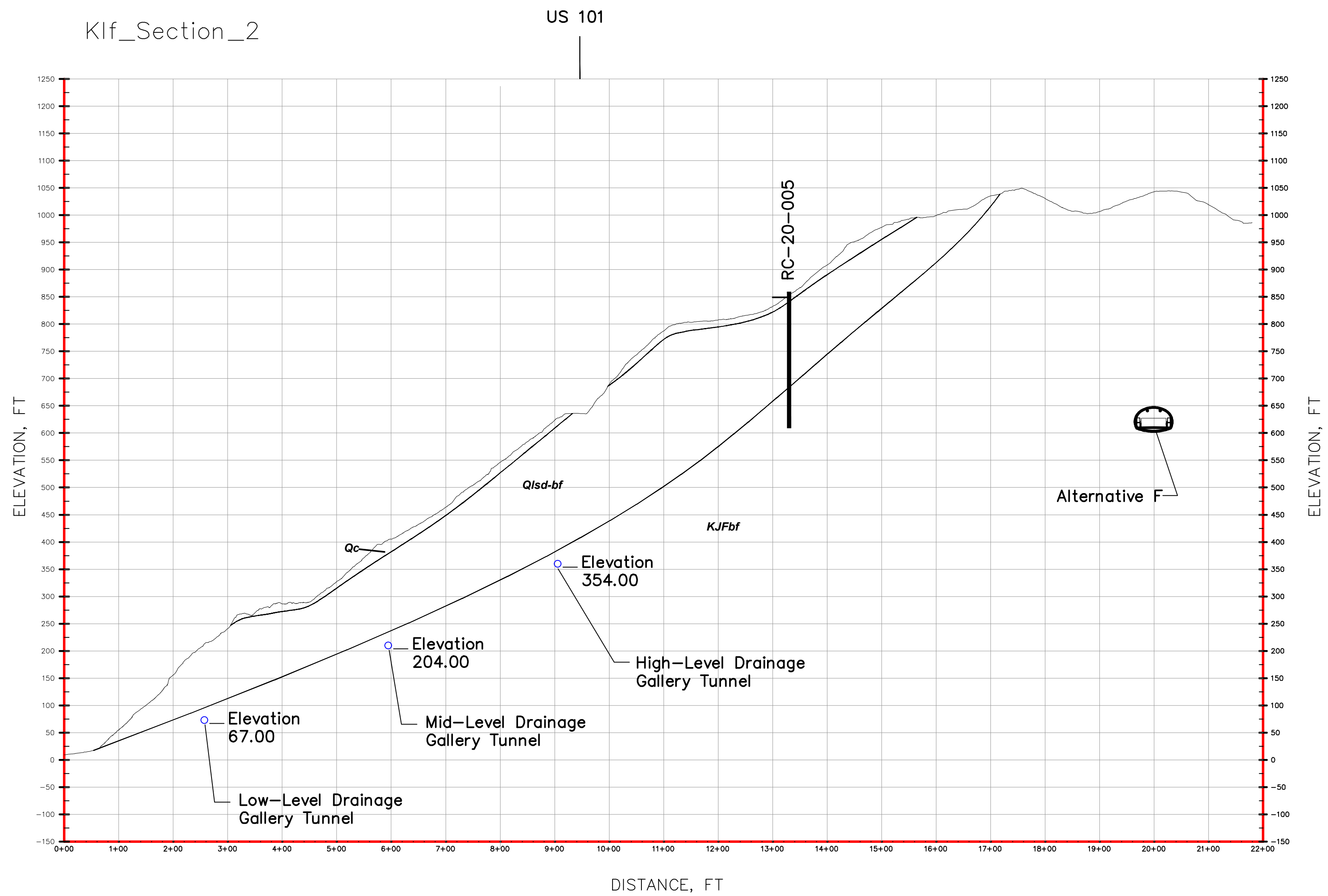


Notes:
1. Data source: Final Preliminary Geotechnical Data Report (Caltrans, 2022a).
2. Design and alignments shown for Alternatives X and F are current as of October 2023.
3. Geology shown is interpreted and based on limited available information. Actual subsurface conditions may differ.
4. Cross sections are intended to illustrate geology and are not intended for detailed design or construction.
5. See Plate 6 for key to geologic symbols.

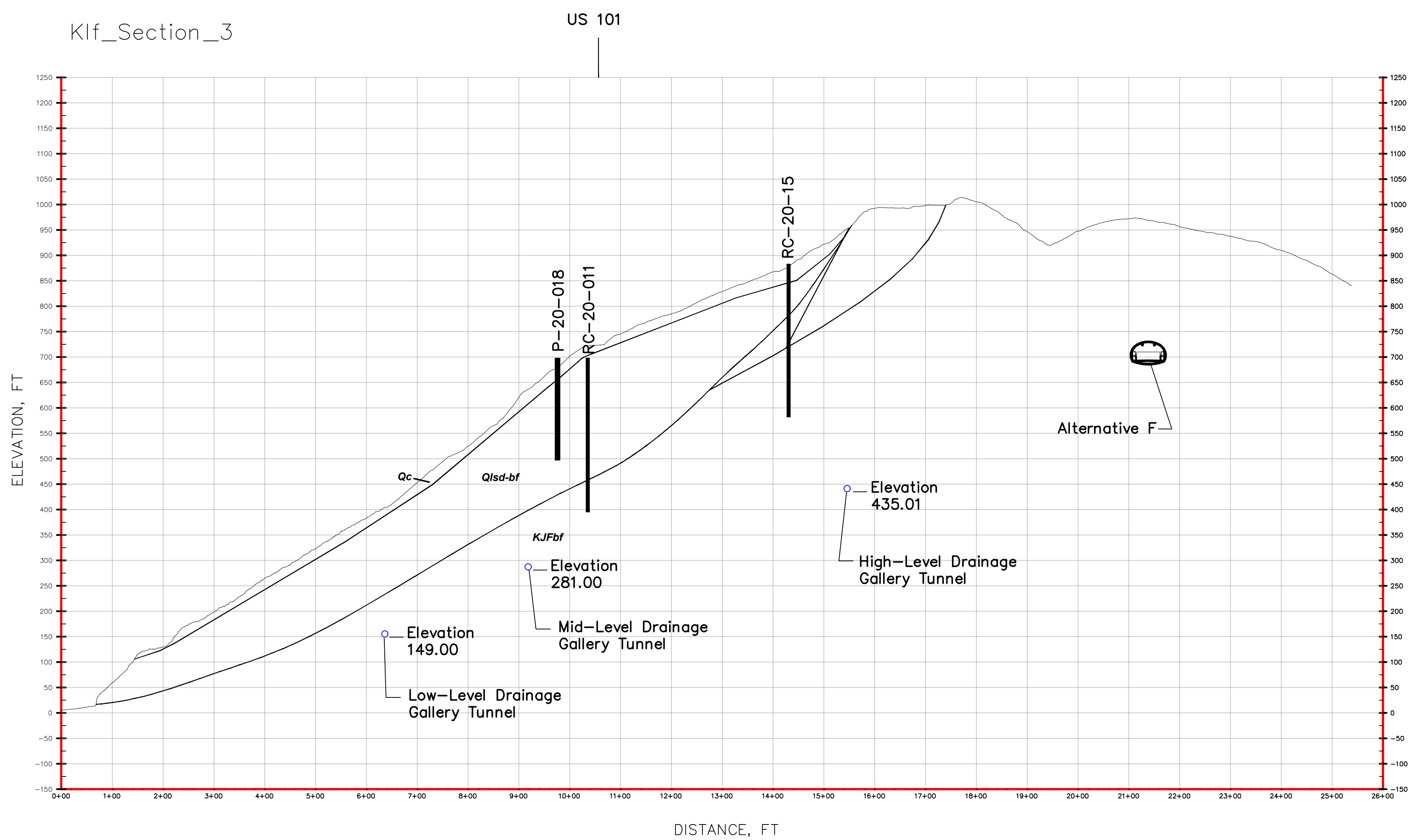
LAST CHANCE GRADE PERMANENT RESTORATION PROJECT PRELIMINARY GEOTECHNICAL REPORT - FINAL	**GEOLOGIC CROSS SECTION 1**	FIGURE **9a**



VIEW: NORTH

Notes:
1. Data source: Final Preliminary Geotechnical Data Report (Caltrans, 2022a).
2. Design and alignments shown for Alternatives X and F are current as of October 2023.
3. Geology shown is interpreted and based on limited available information. Actual subsurface conditions may differ.
4. Cross sections are intended to illustrate geology and are not intended for detailed design or construction.
5. See Plate 6 for key to geologic symbols.

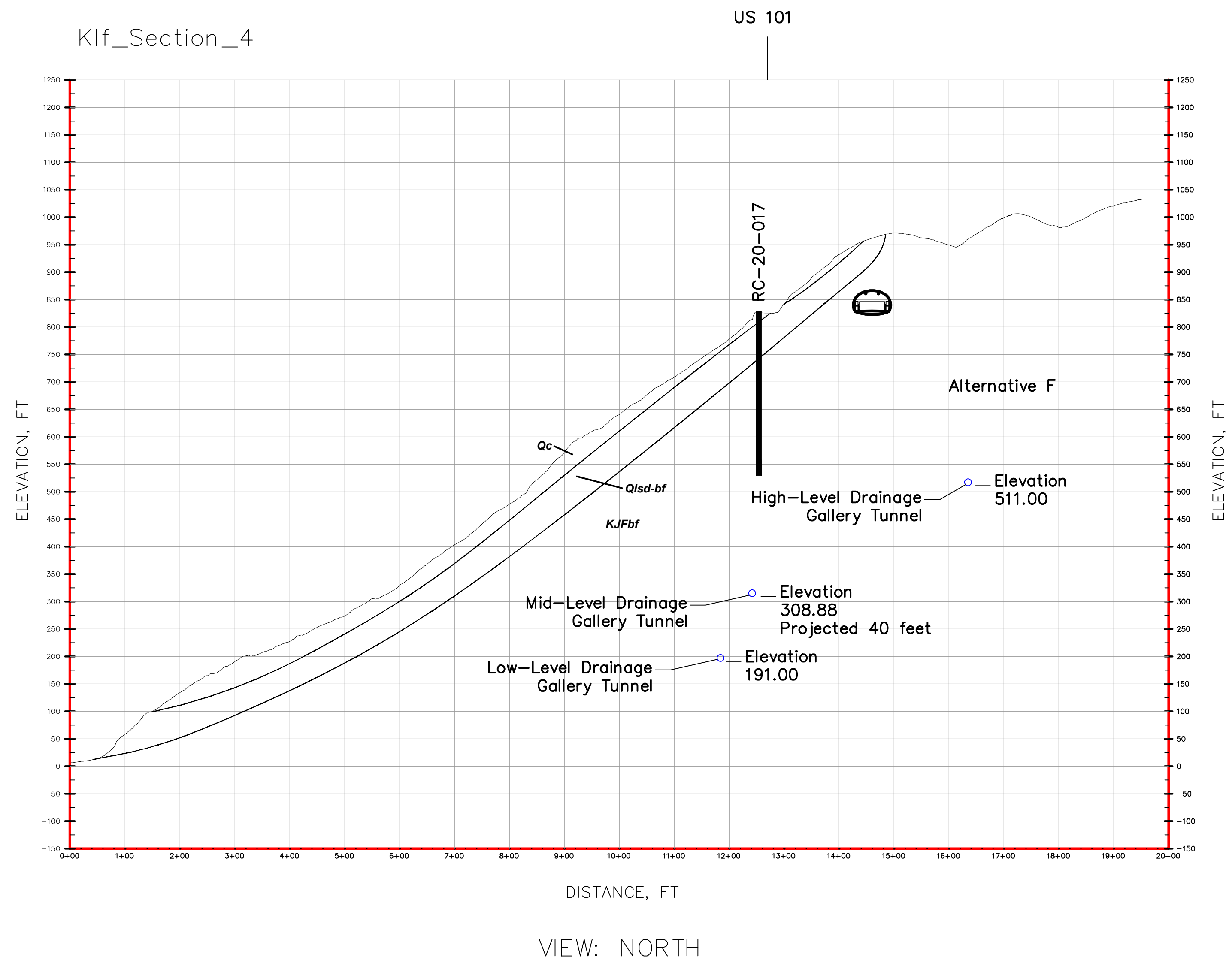
LAST CHANCE GRADE PERMANENT RESTORATION PROJECT PRELIMINARY GEOTECHNICAL REPORT - FINAL	**GEOLOGIC CROSS SECTION 2**	FIGURE **9b**



VIEW: NORTH

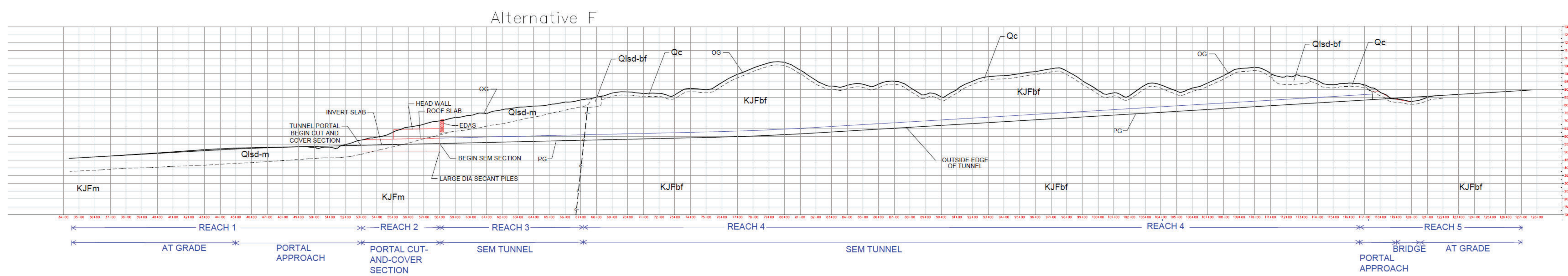
Notes:
1. Data source: Final Preliminary Geotechnical Data Report (Caltrans, 2022a).
2. Design and alignments shown for Alternatives X and F are current as of October 2023.
3. Geology shown is interpreted and based on limited available information. Actual subsurface conditions may differ.
4. Cross sections are intended to illustrate geology and are not intended for detailed design or construction.
5. See Plate 6 for key to geologic symbols.

LAST CHANCE GRADE PERMANENT RESTORATION PROJECT PRELIMINARY GEOTECHNICAL REPORT - FINAL	GEOLOGIC CROSS SECTION 3	FIGURE 9c



Notes:
1. Data source: Final Preliminary Geotechnical Data Report (Caltrans, 2022a).
2. Design and alignments shown for Alternatives X and F are current as of October 2023.
3. Geology shown is interpreted and based on limited available information. Actual subsurface conditions may differ.
4. Cross sections are intended to illustrate geology and are not intended for detailed design or construction.
5. See Plate 6 for key to geologic symbols.

LAST CHANCE GRADE PERMANENT RESTORATION PROJECT PRELIMINARY GEOTECHNICAL REPORT - FINAL	GEOLOGIC CROSS SECTION 4	FIGURE **9d**



LAST CHANCE GRADE PERMANENT RESTORATION PROJECT
PRELIMINARY GEOTECHNICAL REPORT - FINAL

ALTERNATIVE F TUNNEL GEOLOGIC PROFILE

PLATE
10

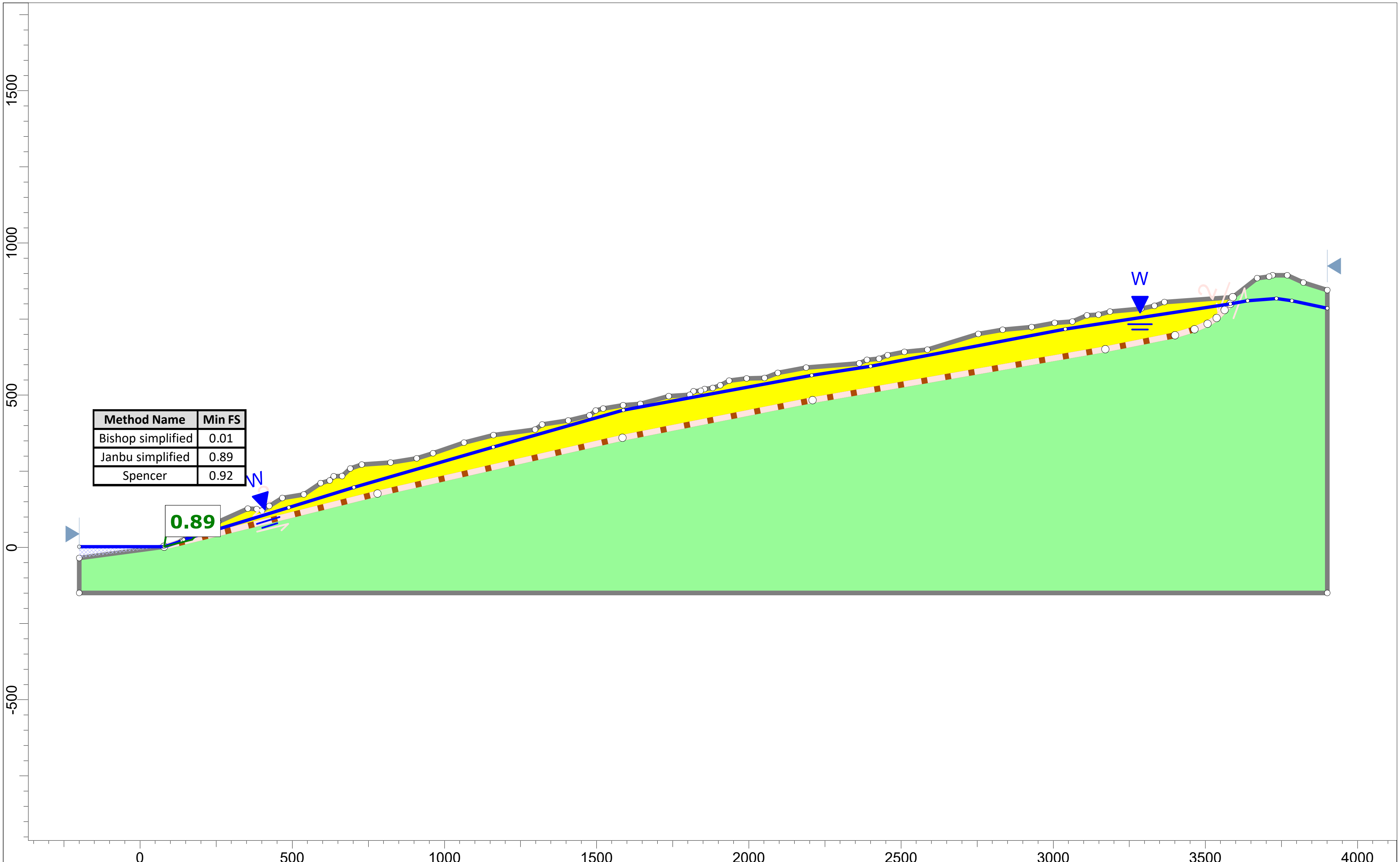
PLOTTED: 06 Nov 2023, 1:52pm, JSala

CAD FILE: W:\2024\20208000.001A - Last Chance Grade\ LAYOUT: Layout1

APPENDICES

APPENDIX A Slope Stability Analyses

APPENDIX A1 Section 1 Earthflow



Method Name	Min FS
Bishop simplified	0.01
Janbu simplified	0.89
Spencer	0.92

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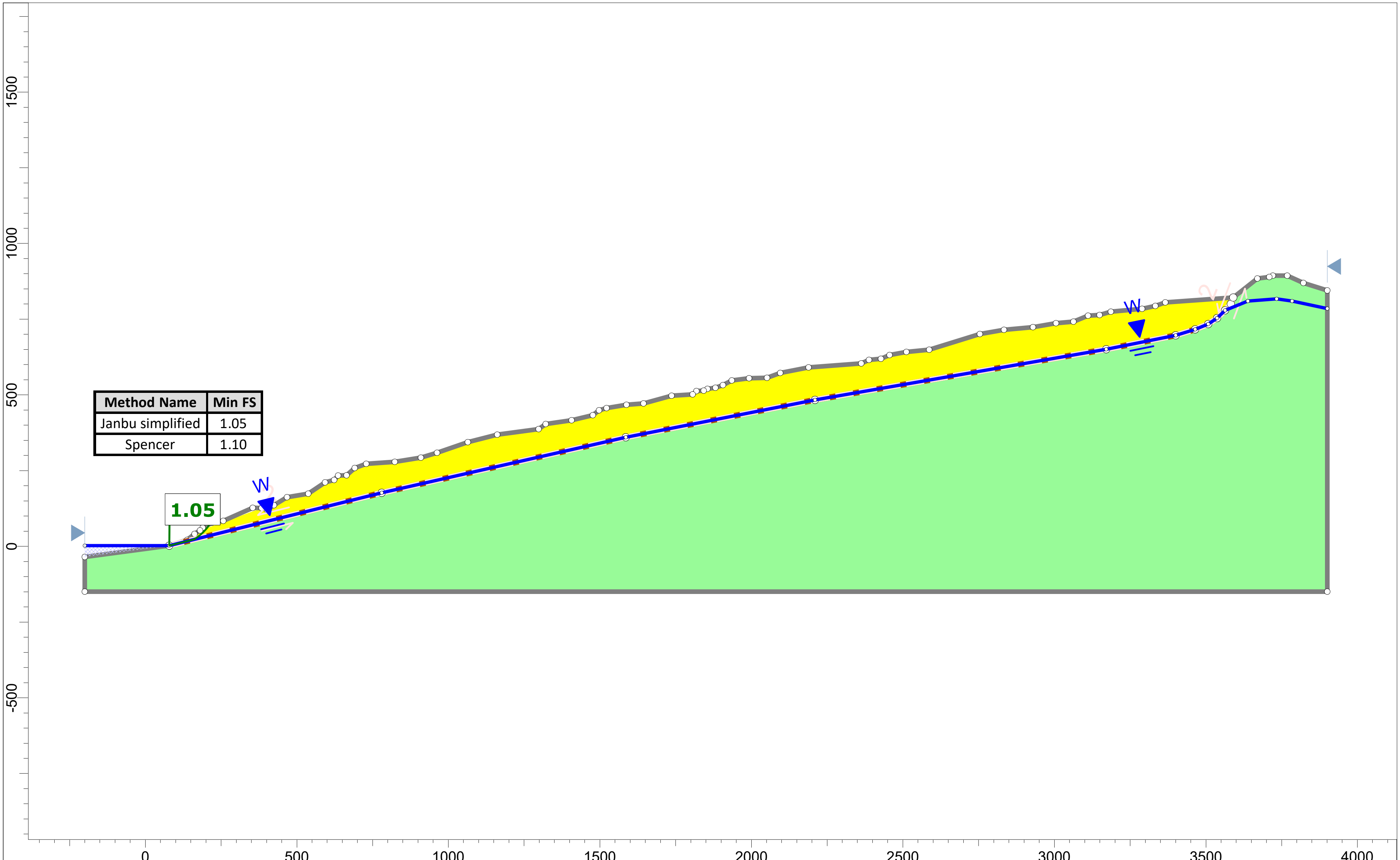
Project No.:	TO C Section 1.slmd - Existing Condition- Wet, Static		**Figure**
Drawn by: Renie Yuen			
Date:11/8/2021			**A-1-1**
Checked by:	**Last Chance Grade**		
Date: 11/8/2021			



Method Name	Min FS
Janbu simplified	1.05
Spencer	1.10

KLEINFELDER
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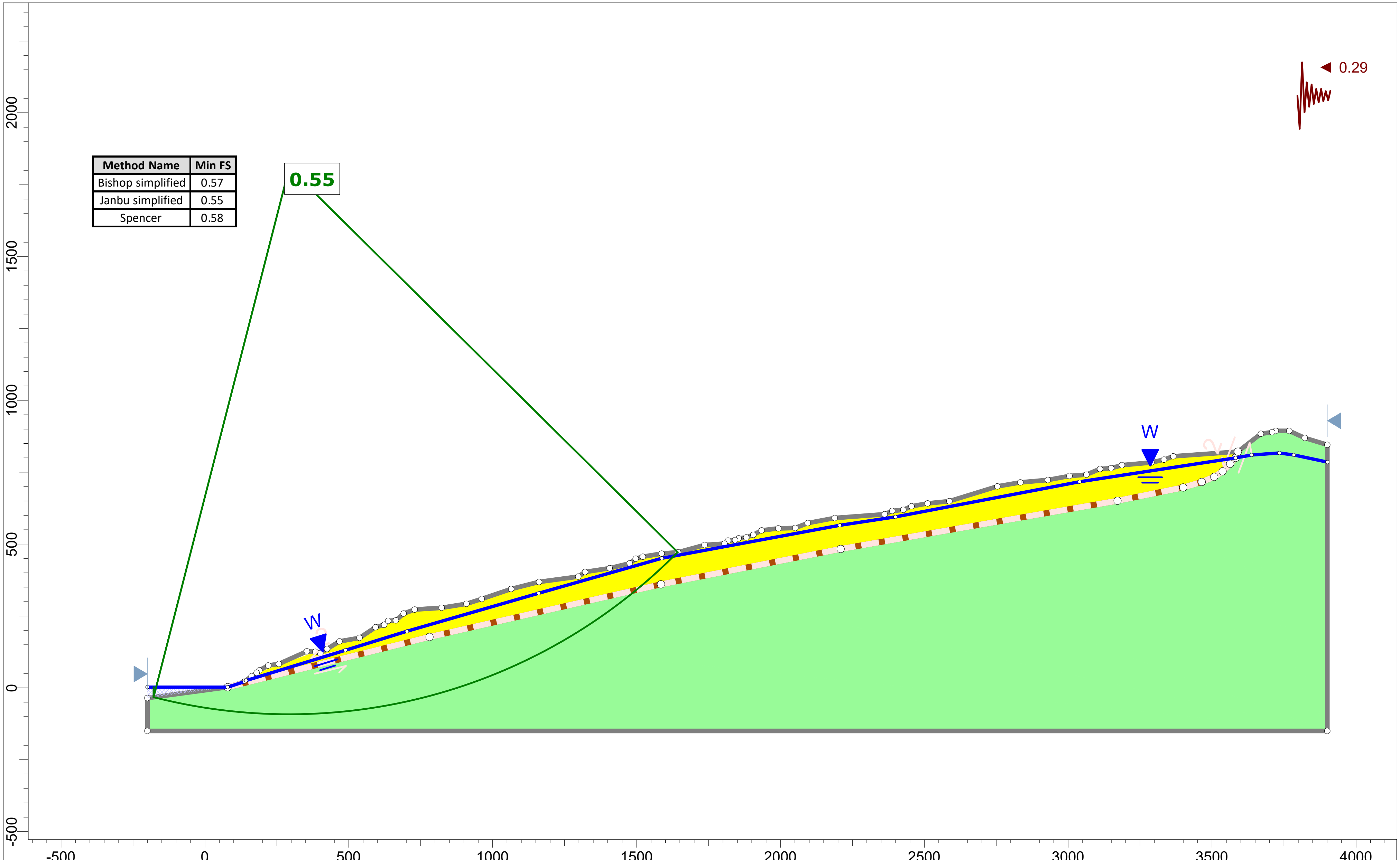
Project No.:	TO C Section 1.slmd - Existing Condition - Dewatered, Static
Drawn by: Renie Yuen	
Date:11/8/2021	
Checked by:	Last Chance Grade
Date: 11/8/2021	

Figure

A-1-2



Method Name	Min FS
Bishop simplified	0.57
Janbu simplified	0.55
Spencer	0.58

KLEINFELDER
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Project No.:	TO C Section 1.slmd - Existing Condition- Wet, Pseudostatic	**Figure**
Drawn by: Renie Yuen		**A-1-3**
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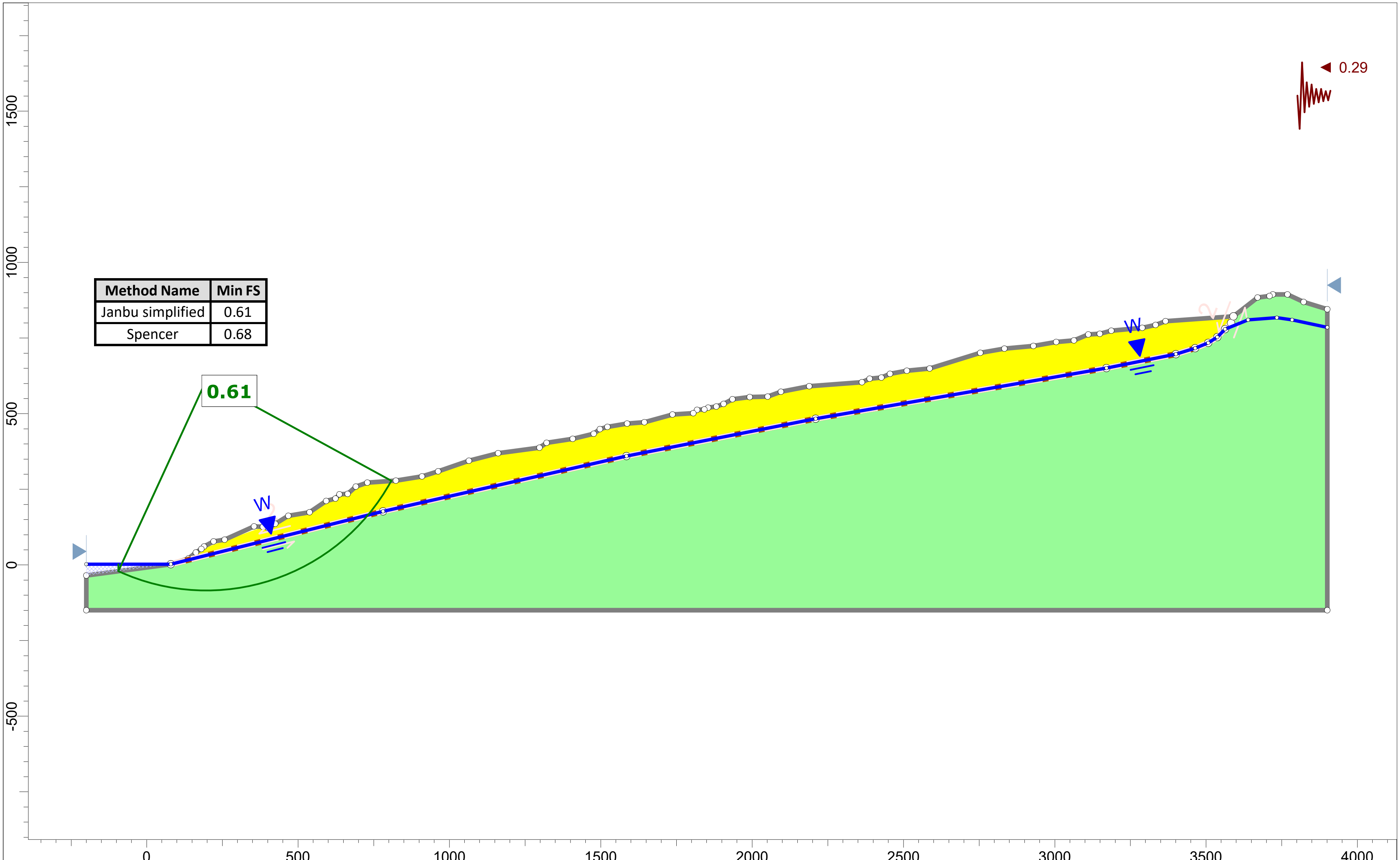
Checked by:	**Last Chance Grade**	
Date: 11/8/2021		



Method Name	Min FS
Janbu simplified	0.61
Spencer	0.68

KLEINFELDER
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Project No.:	TO C Section 1.slmd - Existing Condition - Dewatered, Pseudostatic	**Figure**
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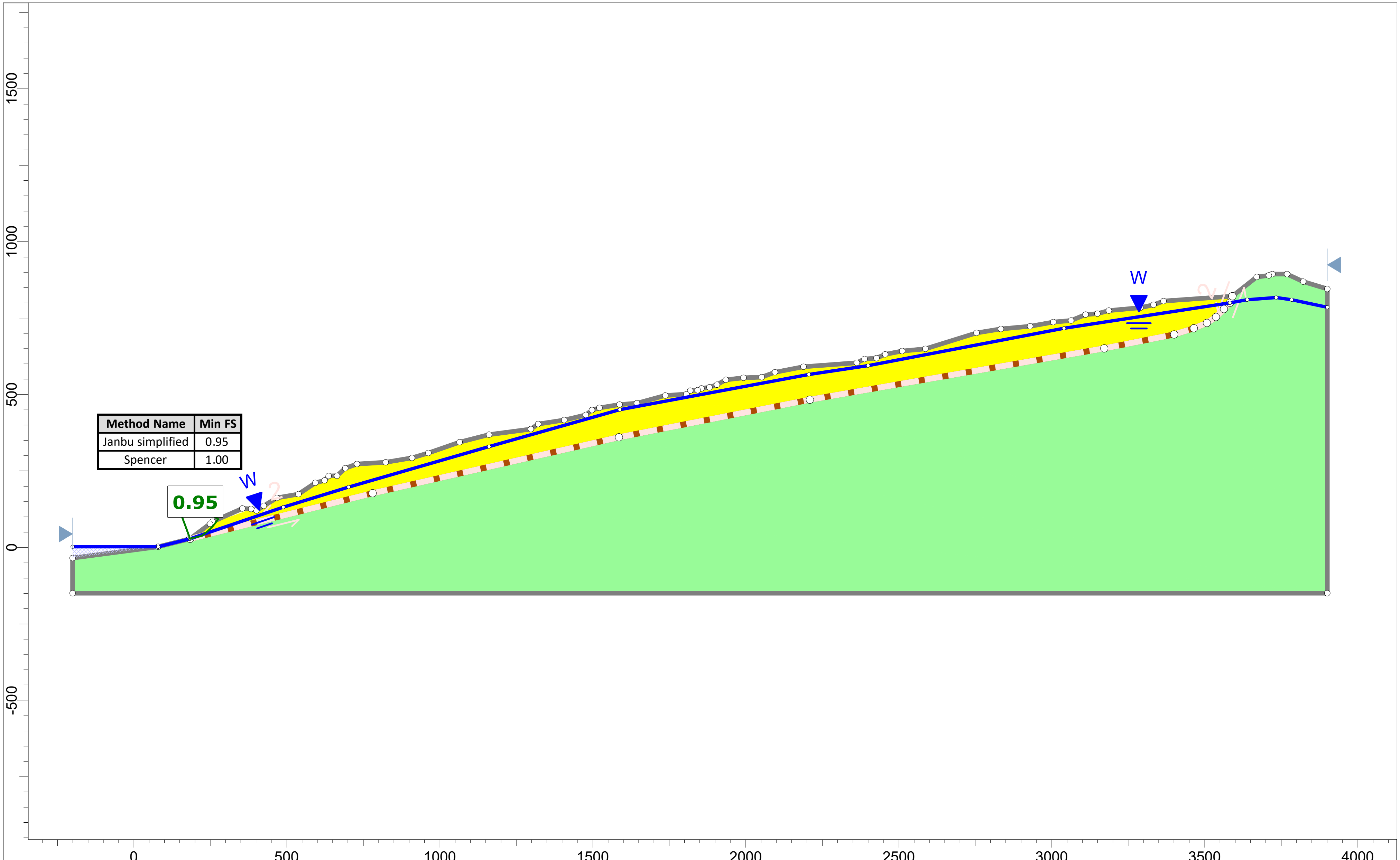
Checked by:	**Last Chance Grade**	
Date: 11/8/2021		



KLEINFELDER *Bright People. Right Solutions.*	Project No.:	TO C Section 1.slmd - Coastal Erosion - Wet, Static	**Figure**
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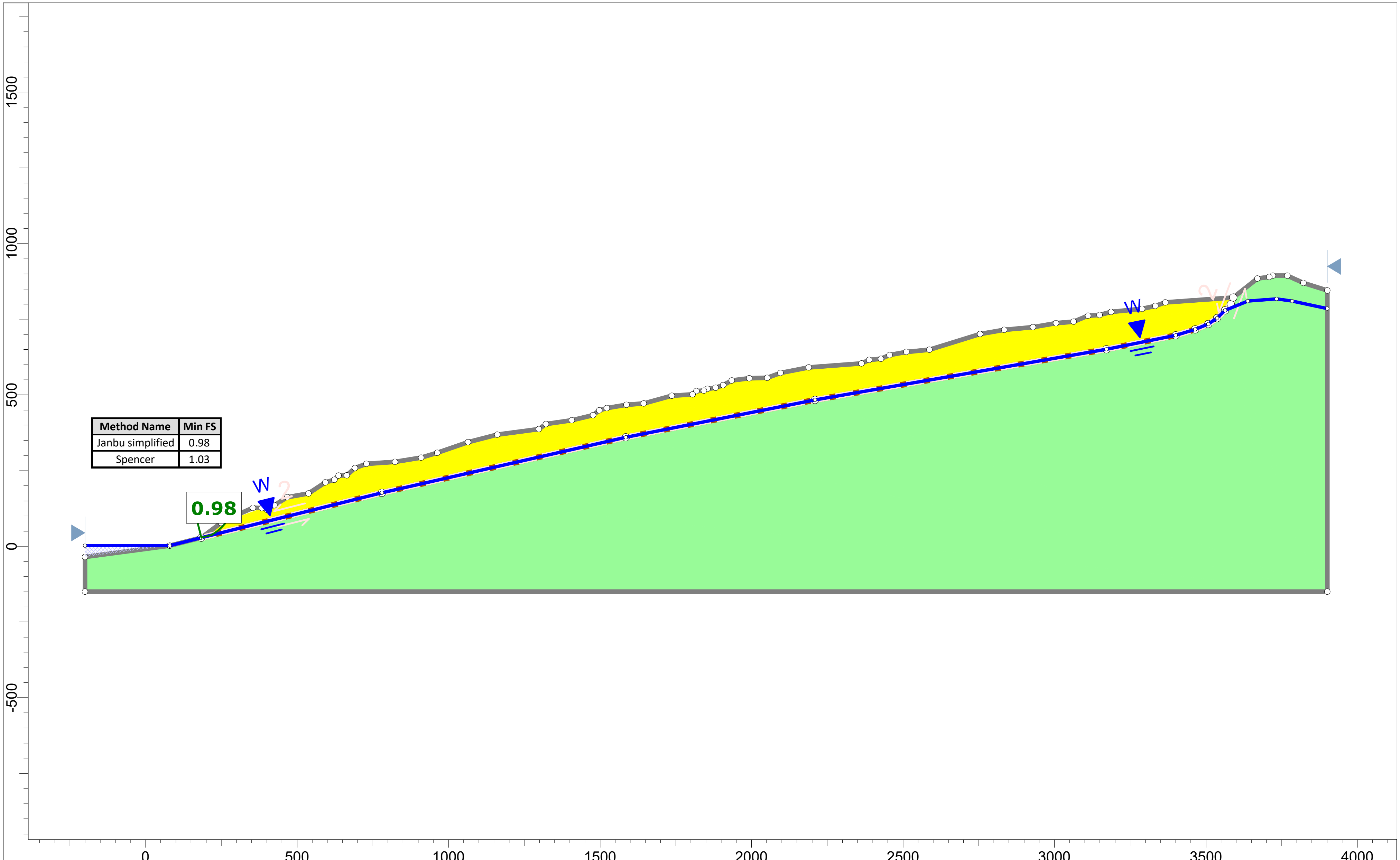
	Date: 11/8/2021		



Method Name	Min FS
Janbu simplified	0.98
Spencer	1.03

KLEINFELDER
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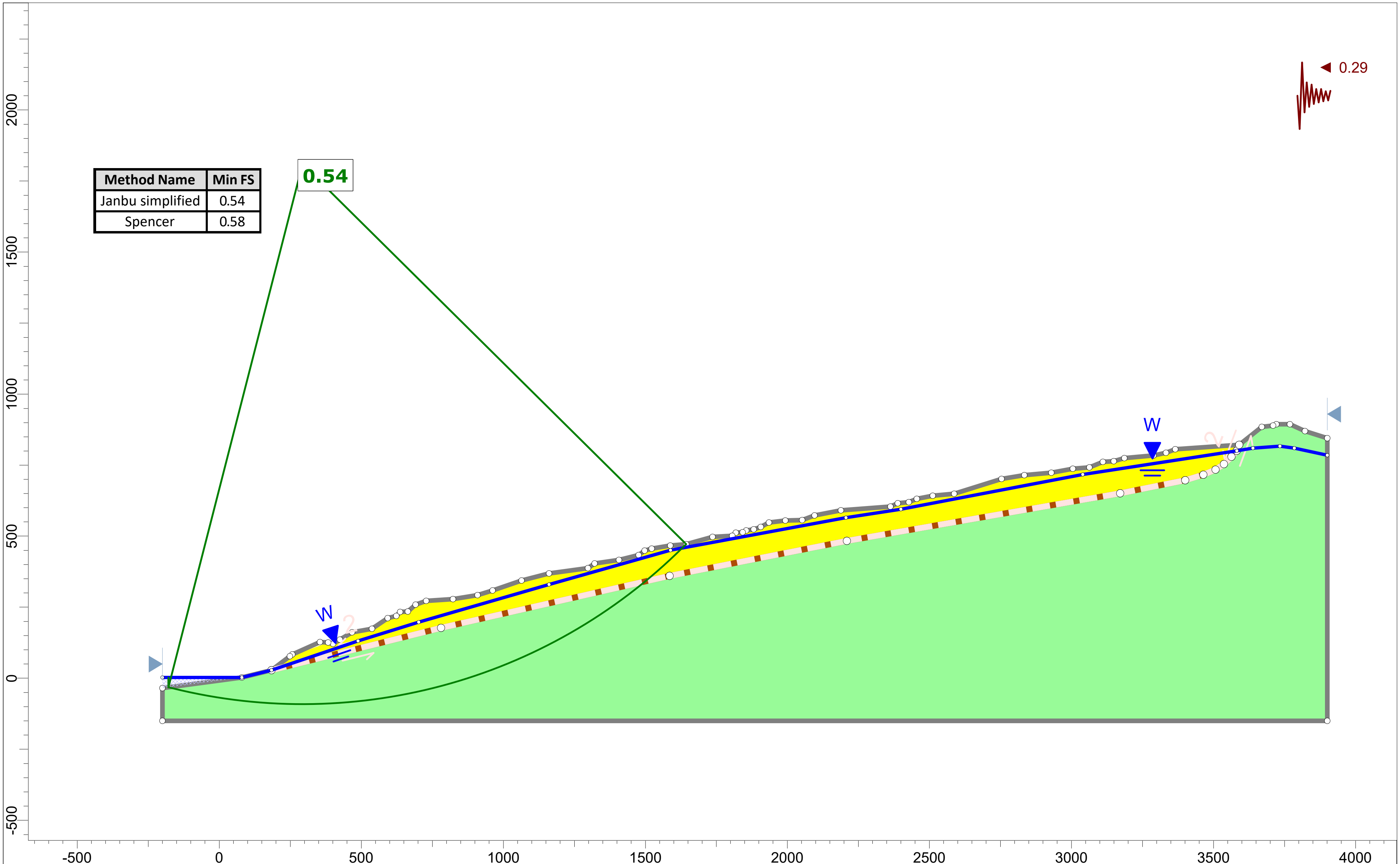
Project No.:	TO C Section 1.slmd - Coastal Erosion - Dewatered, Static
Drawn by: Renie Yuen	
Date:11/8/2021	
Checked by:	Last Chance Grade
Date: 11/8/2021	

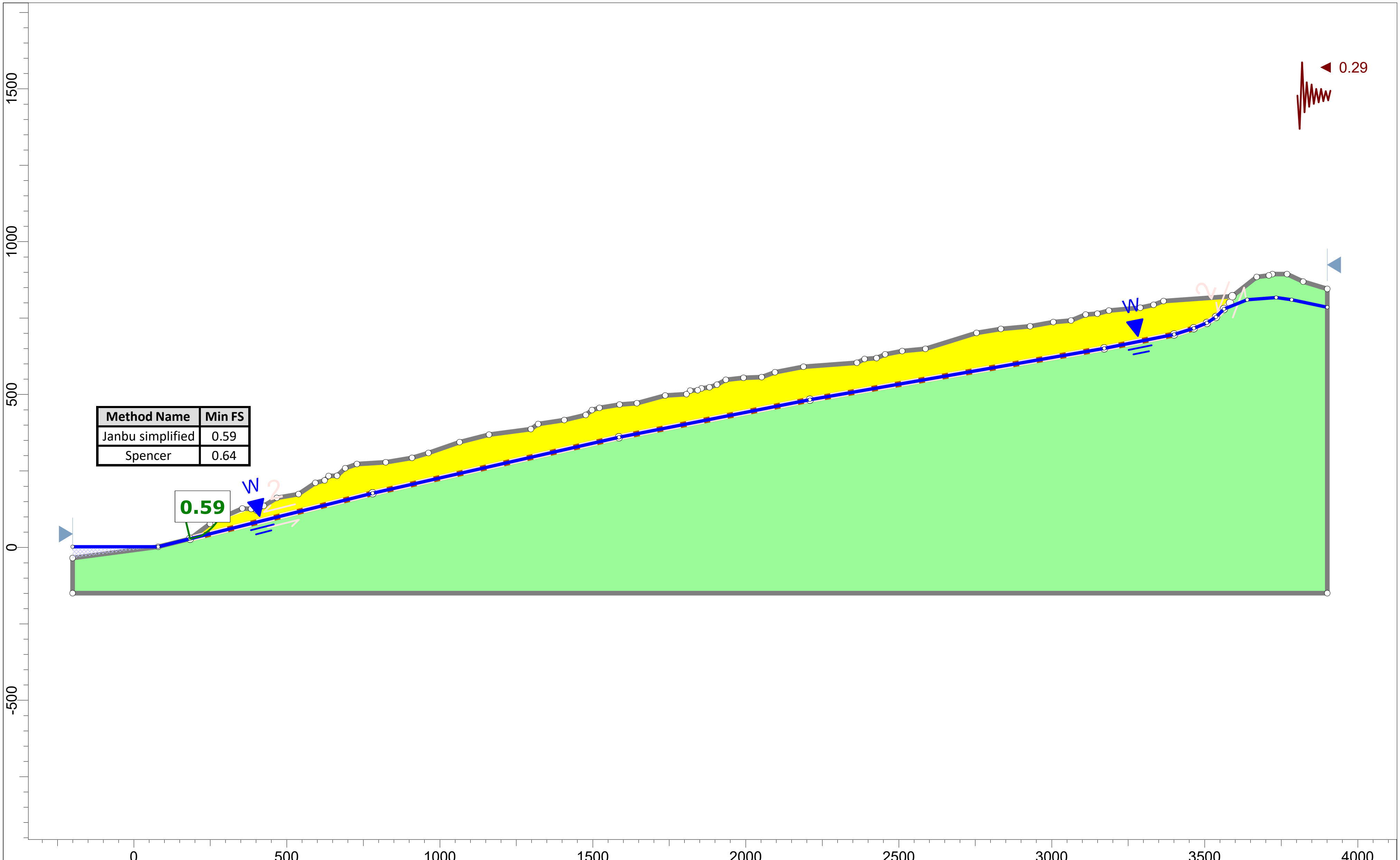
Figure

A-1-6



KLEINFELDER
Bright People. Right Solutions.

Project No.:	TO C Section 1.slmd - Coastal Erosion - Wet, Pseudostatic	**Figure**
Drawn by: Renie Yuen		**A-1-7**
Date:11/8/2021		
Checked by:	**Last Chance Grade**	
Date: 11/8/2021		

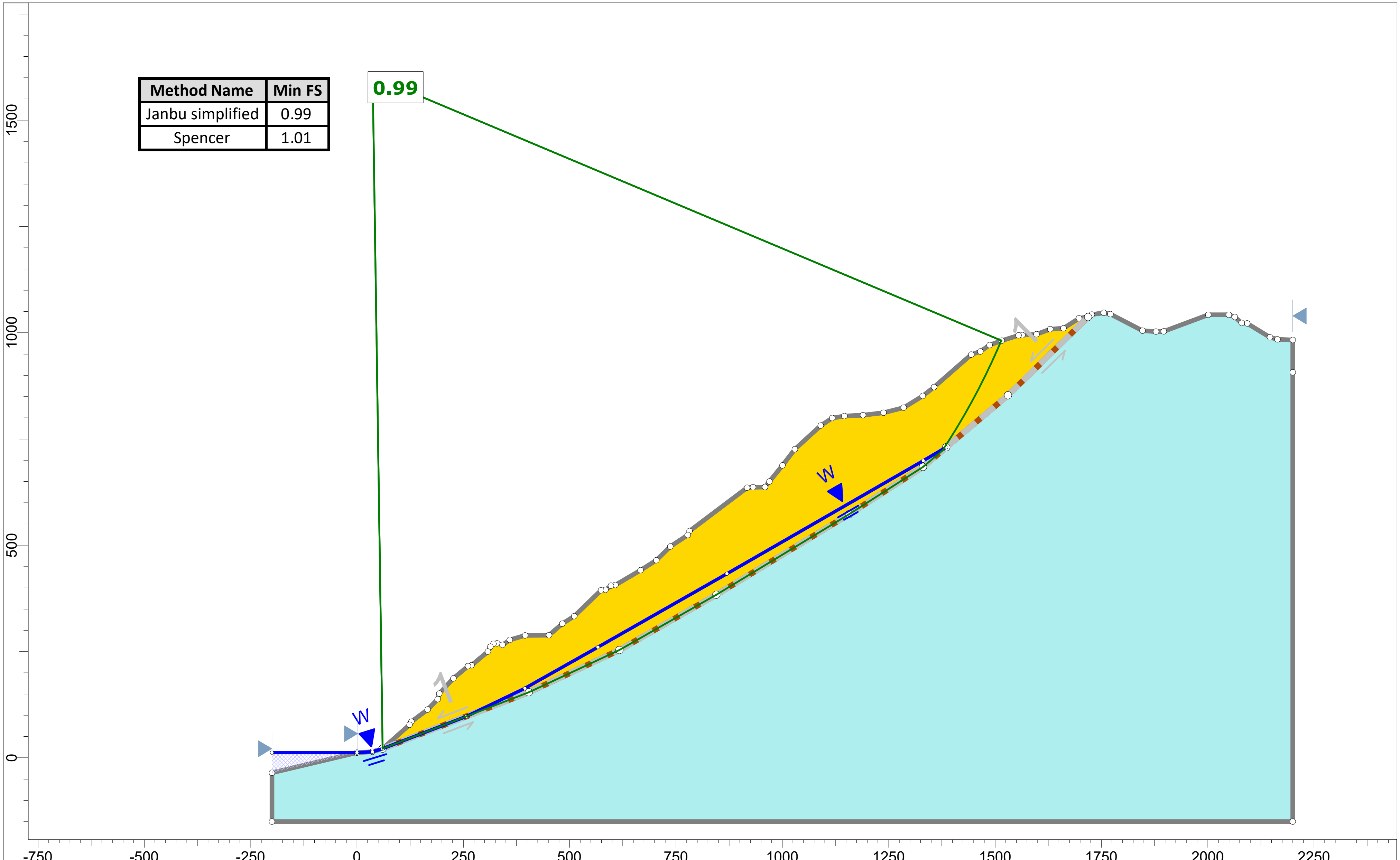


Method Name	Min FS
Janbu simplified	0.59
Spencer	0.64

KLEINFELDER
Bright People. Right Solutions.

Project No.:	TO C Section 1.slmd - Coastal Erosion - Dewatered, Pseudostatic		**Figure**
Drawn by: Renie Yuen			**A-1-8**
Date:11/8/2021			
Checked by:	**Last Chance Grade**		
Date: 11/8/2021			

APPENDIX A2 Section 2 Wilson Creek Complex



KLEINFELDER Bright People. Right Solutions.	Project No.:	TO C Section 2.slmd - Existing Condition - Wet, Static	Figure
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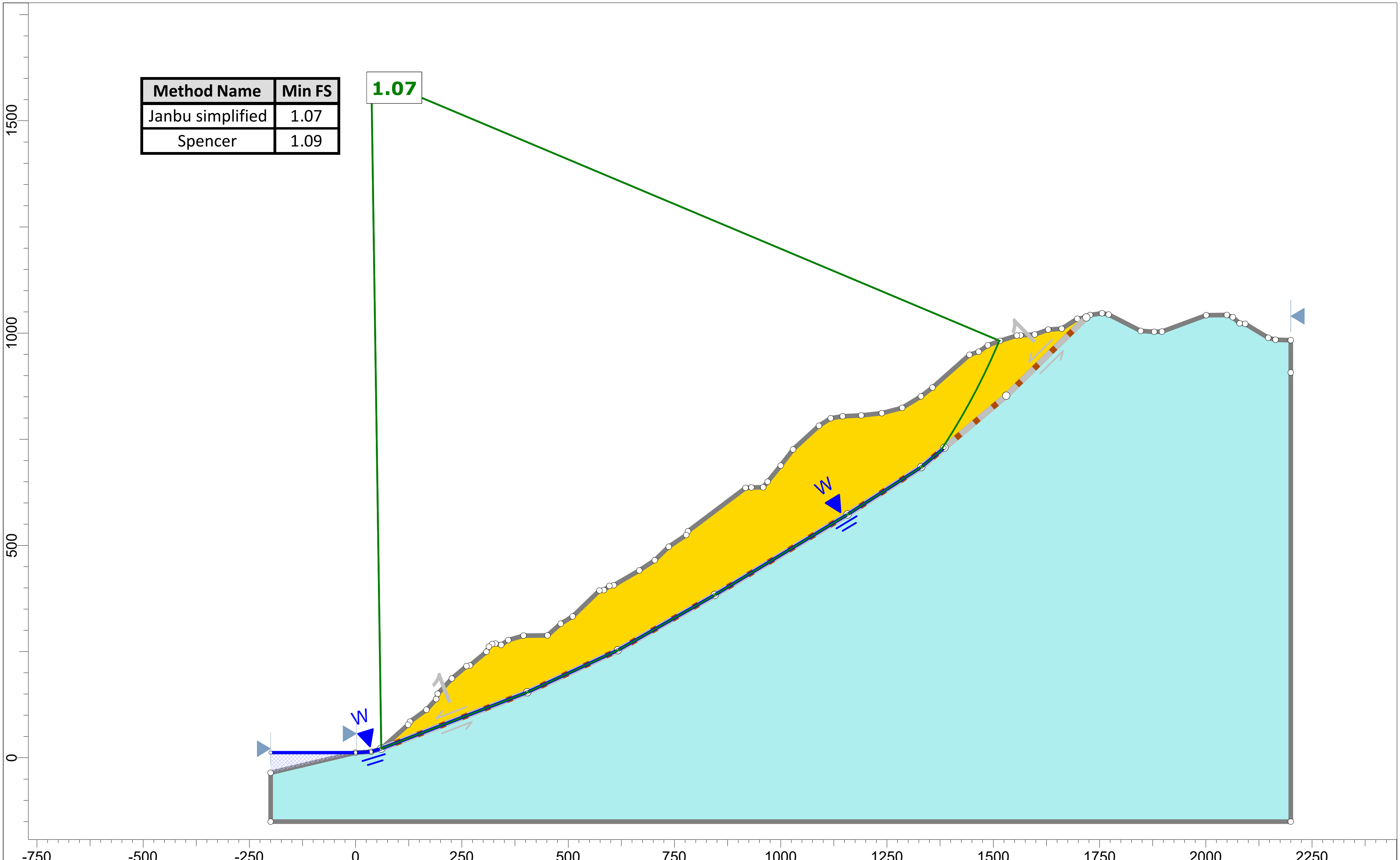

Method Name	Min FS
Janbu simplified	1.07
Spencer	1.09

KLEINFELDER Bright People. Right Solutions.	Project No.:	TO C Section 2.slmd - Existing Condition - Dewatered, Static	**Figure**
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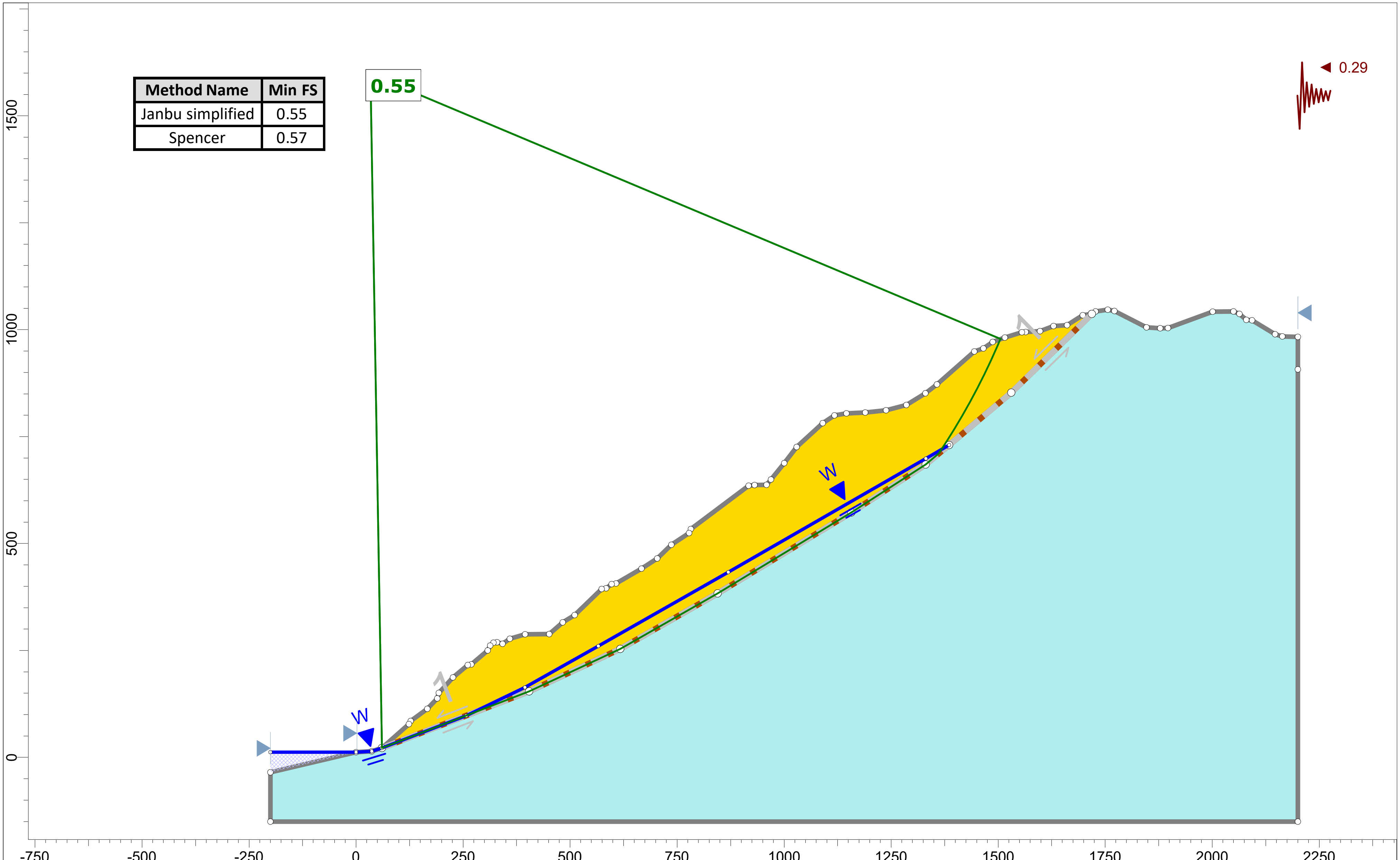
	Date: 11/9/2021		



Method Name	Min FS
Janbu simplified	0.55
Spencer	0.57

KLEINFELDER
Bright People. Right Solutions.

Project No.:	TO C Section 2.slmd - Existing Condition - Wet, Pseudostatic	**Figure**
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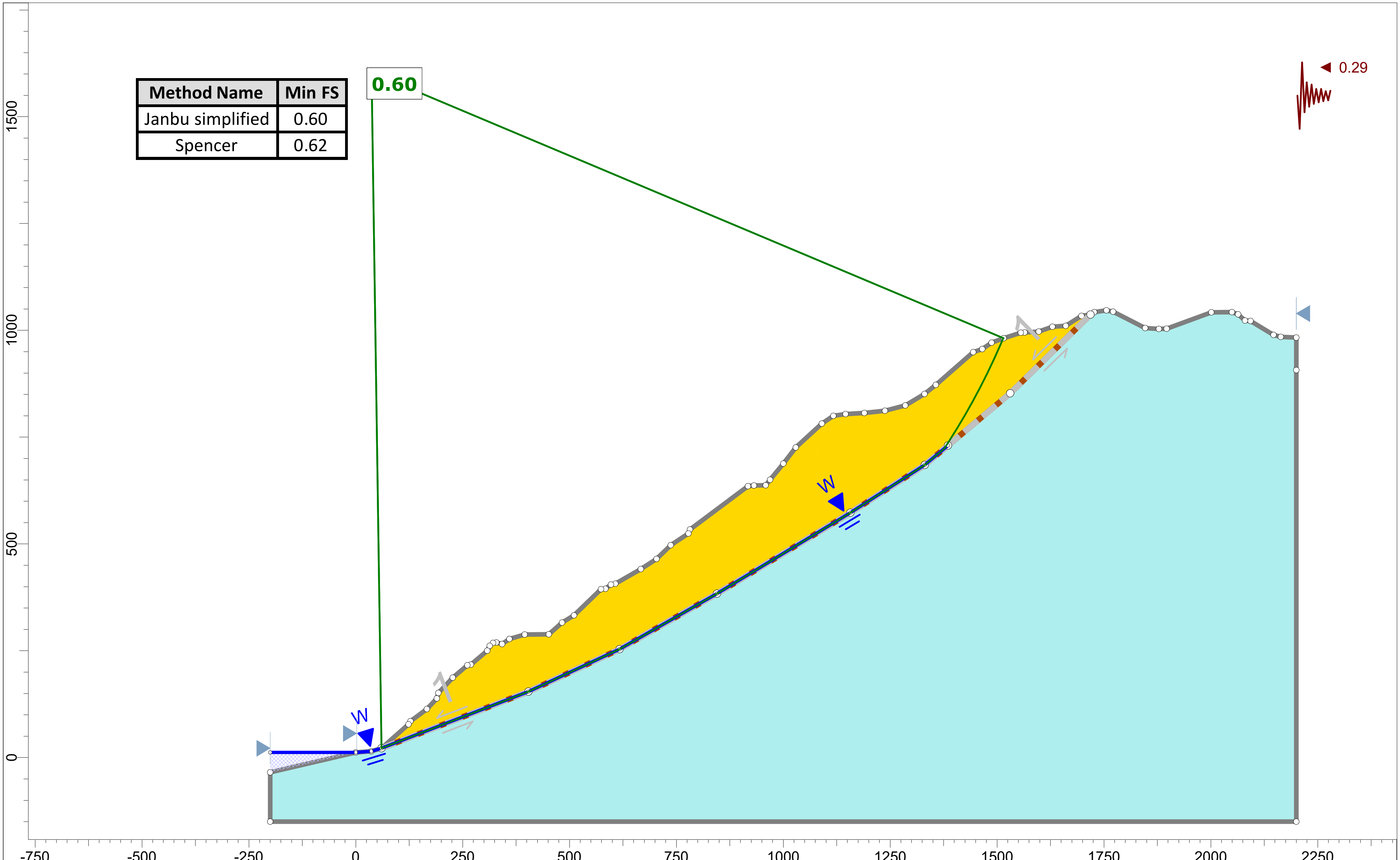
Checked by:	**Last Chance Grade**	
Date: 11/9/2021		



Method Name	Min FS
Janbu simplified	0.60
Spencer	0.62

KLEINFELDER
Bright People. Right Solutions.

Project No.:	TO C Section 2.slmd - Existing Condition - Dewatered, Pseudostatic		**Figure**
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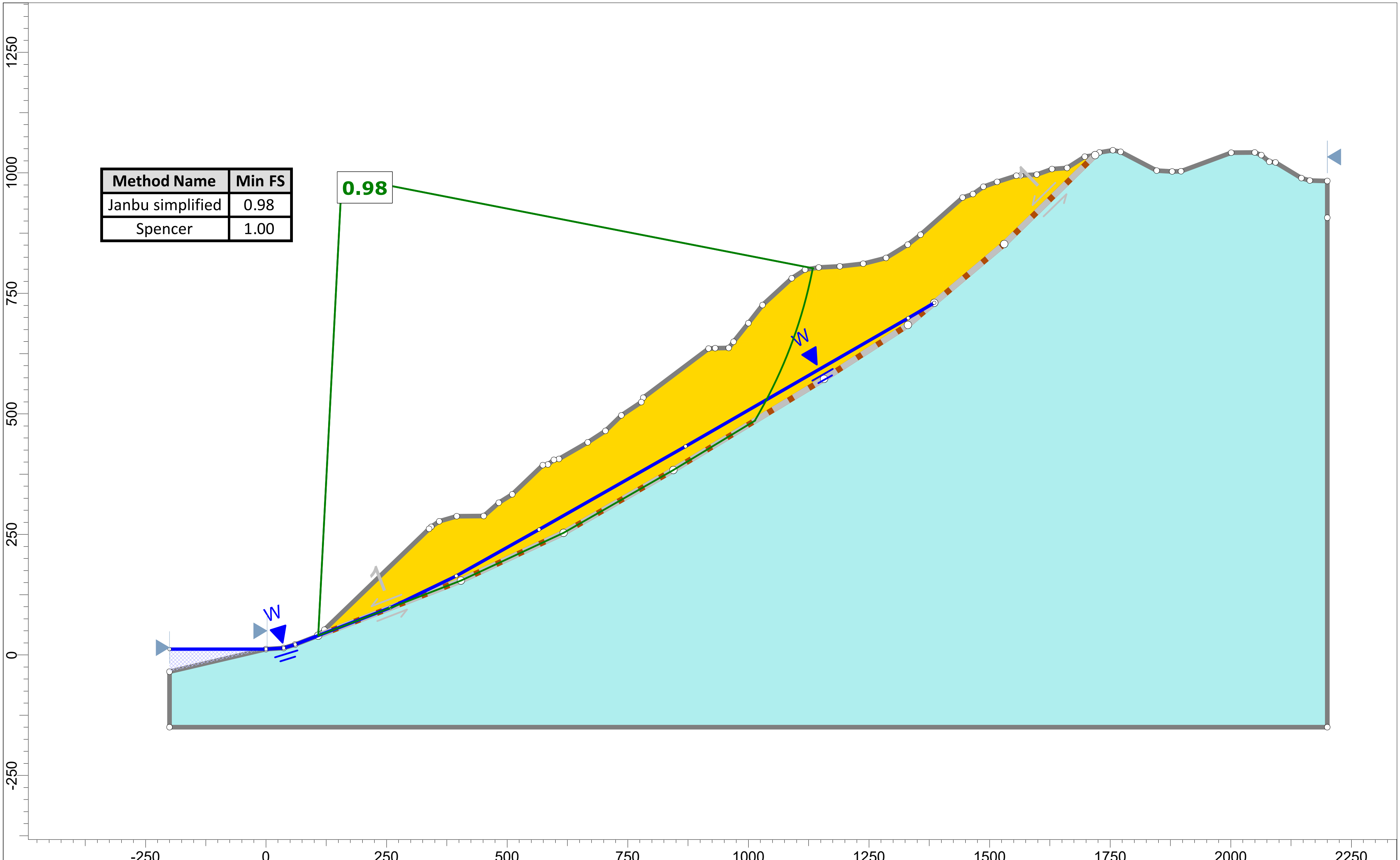
Checked by:	**Last Chance Grade**		
Date: 11/9/2021			



Method Name	Min FS
Janbu simplified	0.98
Spencer	1.00

KLEINFELDER
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Project No.:	TO C Section 2.slmd - Coastal Erosion - Wet, Static	**Figure**
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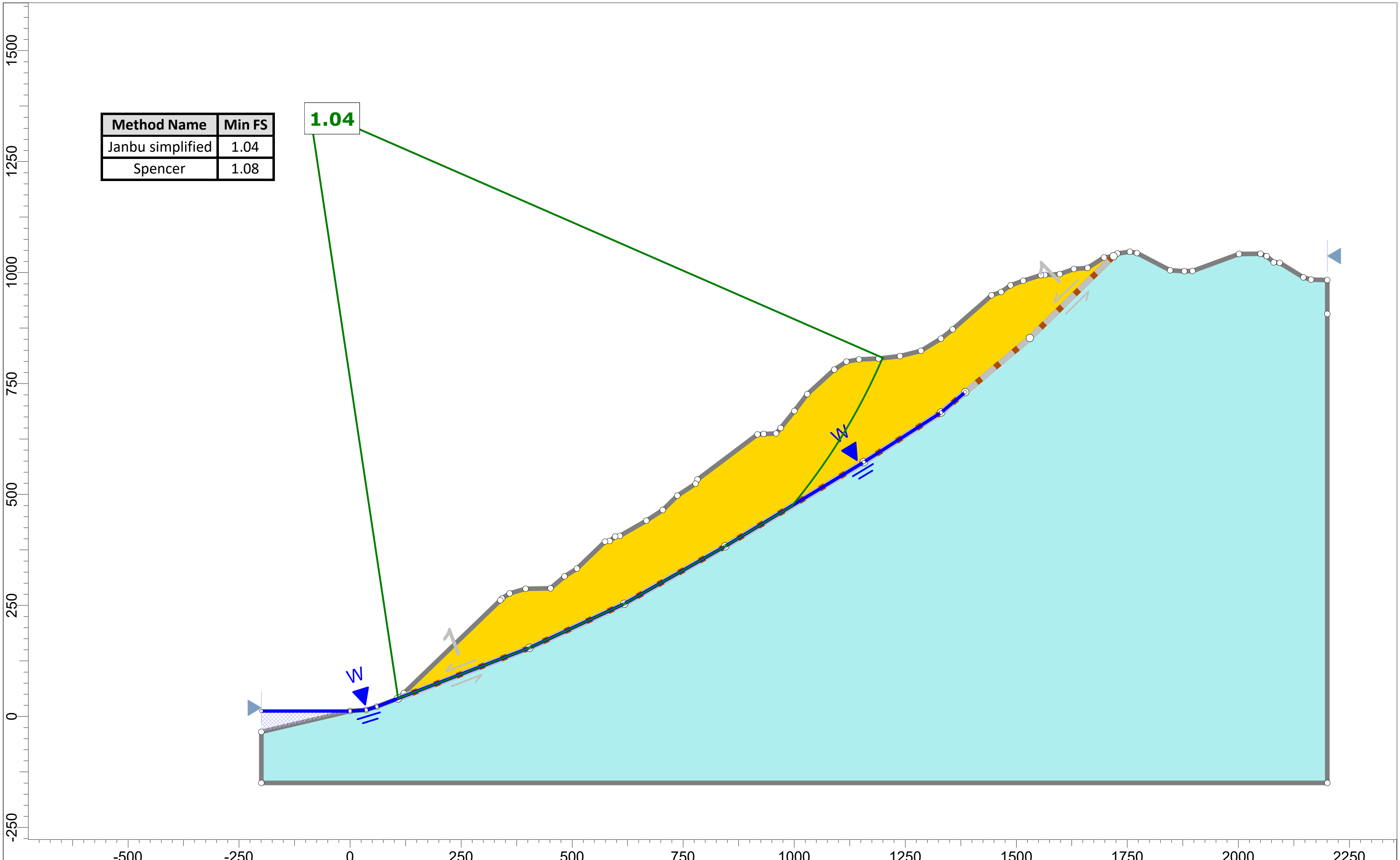
Date:11/9/2021		
Checked by:	**Last Chance Grade**	
Date: 11/9/2021		



KLEINFELDER
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Project No.:	TO C Section 2.slmd - Coastal Erosion - Dewatered, Static	**Figure**
Drawn by: Renie Yuen		**A-2-6**
Date:11/9/2021		
Checked by:	**Last Chance Grade**	
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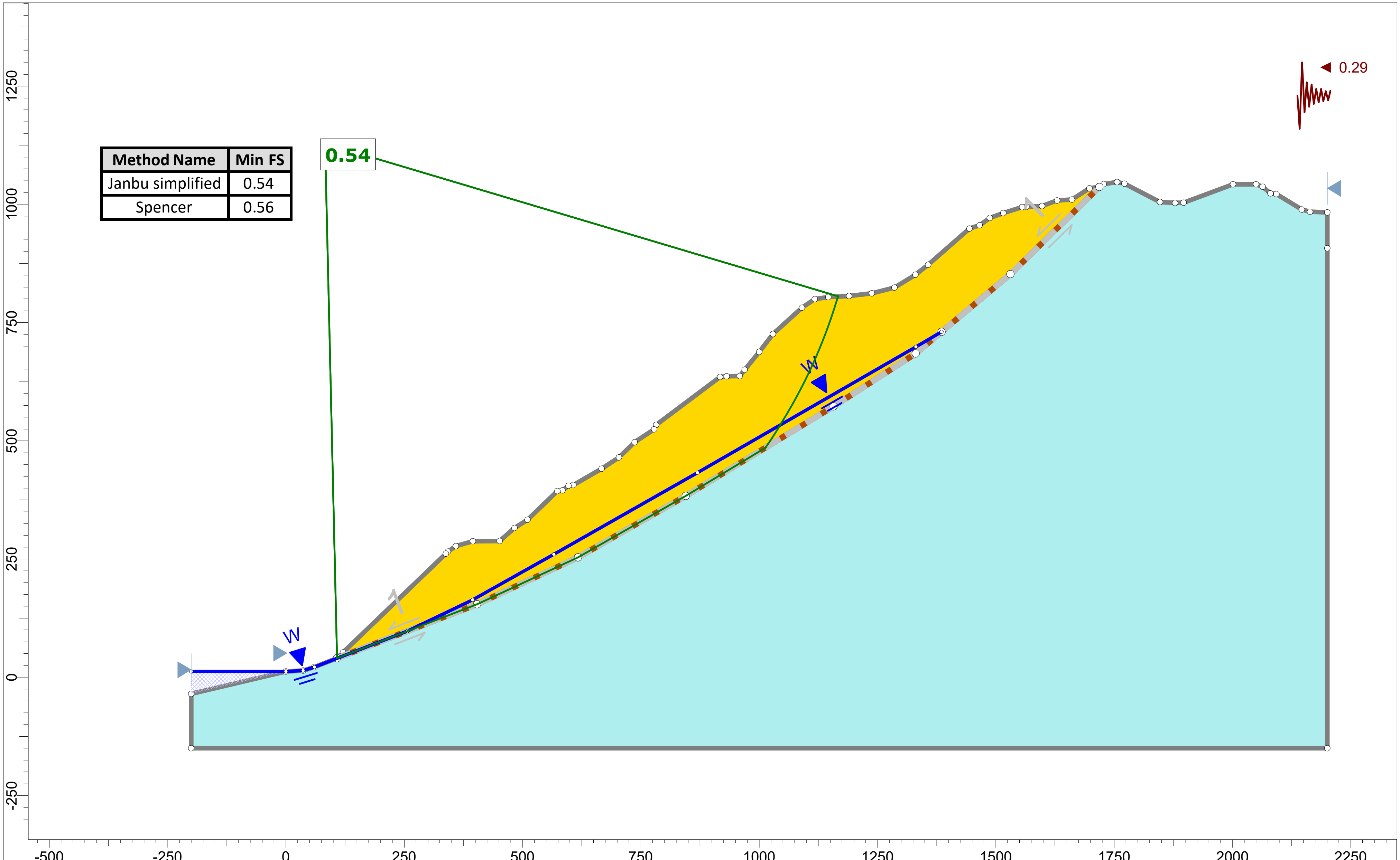

KLEINFELDER
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Project No.:	TO C Section 2.slmd - Coastal Erosion - Wet, Pseudostatic		**Figure**
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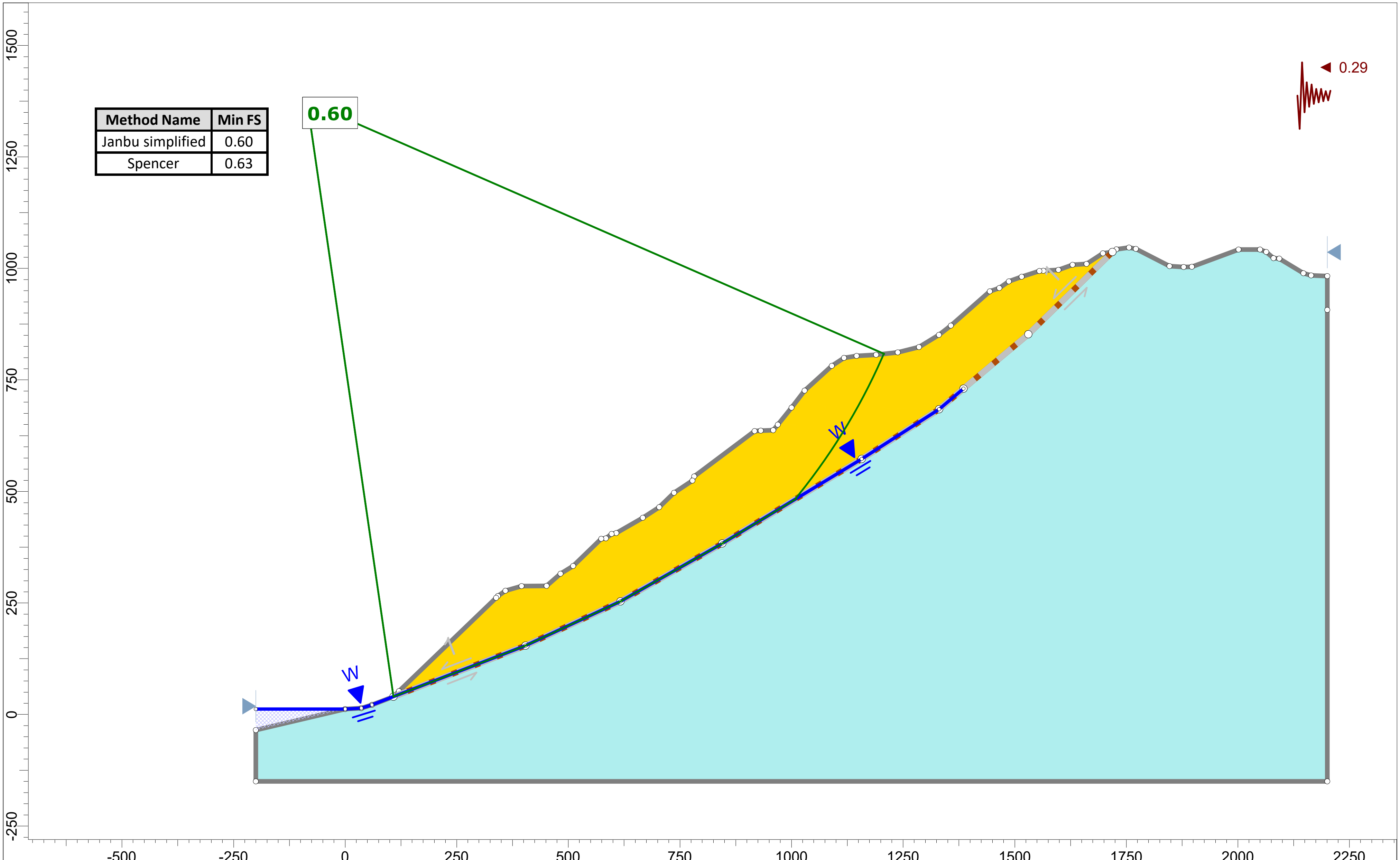
Date:11/9/2021			**A-2-7**
Checked by:	**Last Chance Grade**		
Date: 11/9/2021			



KLEINFELDER
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Project No.:	TO C Section 2.slmd - Coastal Erosion - Dewatered, Pseudostatic	**Figure**
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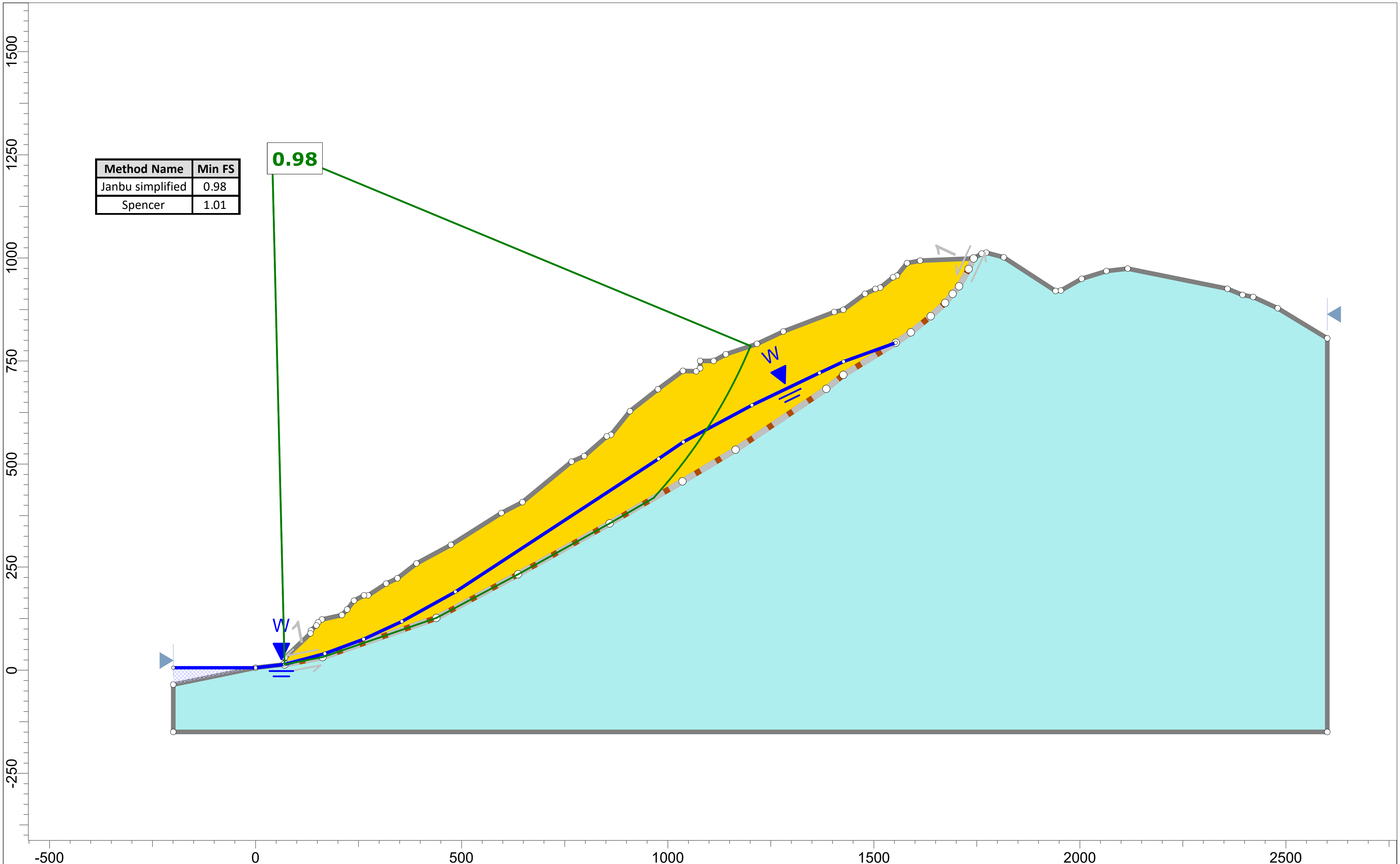
Date:11/9/2021		
Checked by:	**Last Chance Grade**	
Date: 11/9/2021		

APPENDIX A3 Section 3 South LCG Complex



KLEINFELDER
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Project No.:	TO C Section 3.slmd - Existing Condition - Wet, Static		Figure A-3-1
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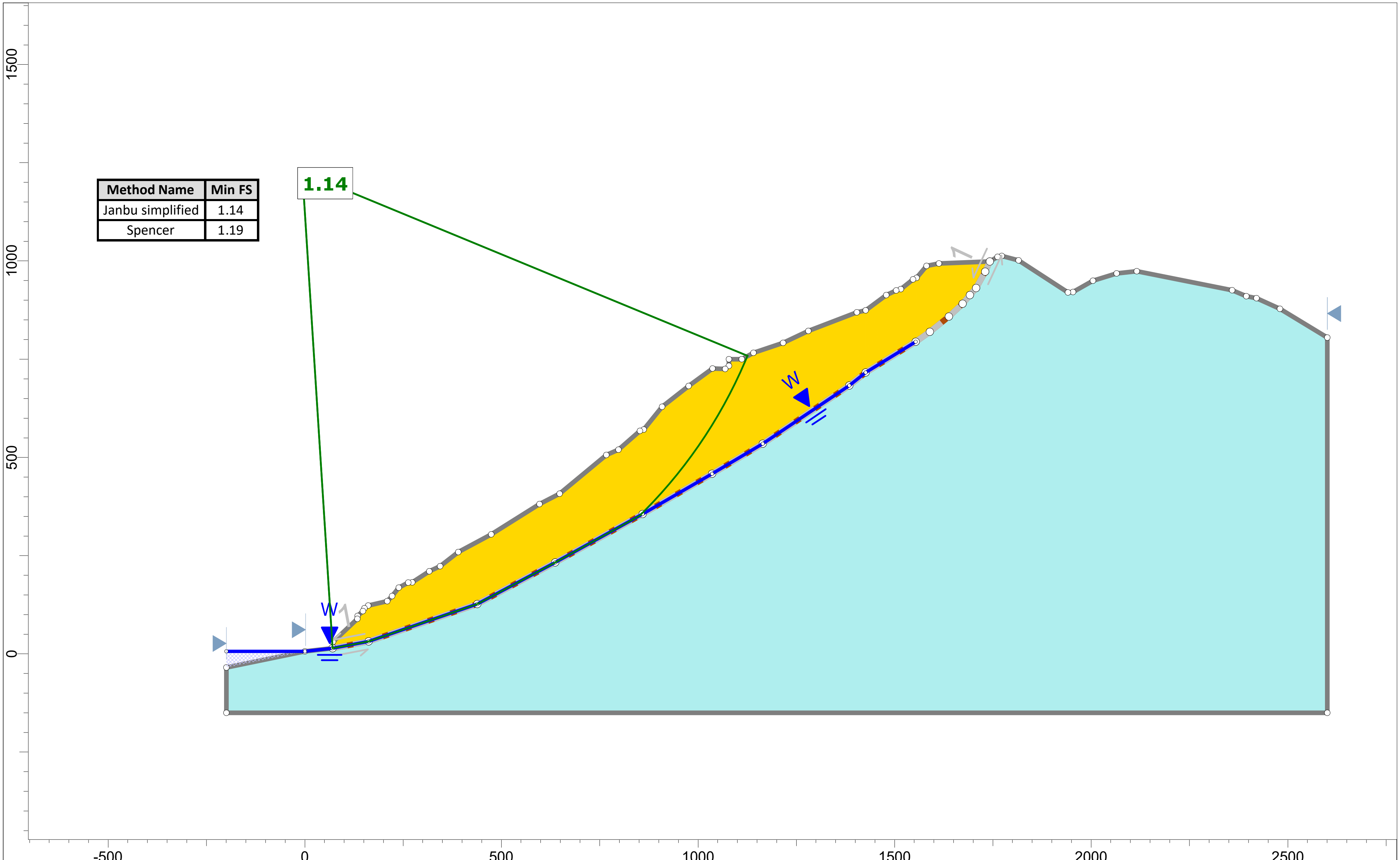
Date: 11/9/2021			



Method Name	Min FS
Janbu simplified	1.14
Spencer	1.19

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Project No.:	TO C Section 3.slmd - Existing Condition - Dewatered, Static		**Figure**
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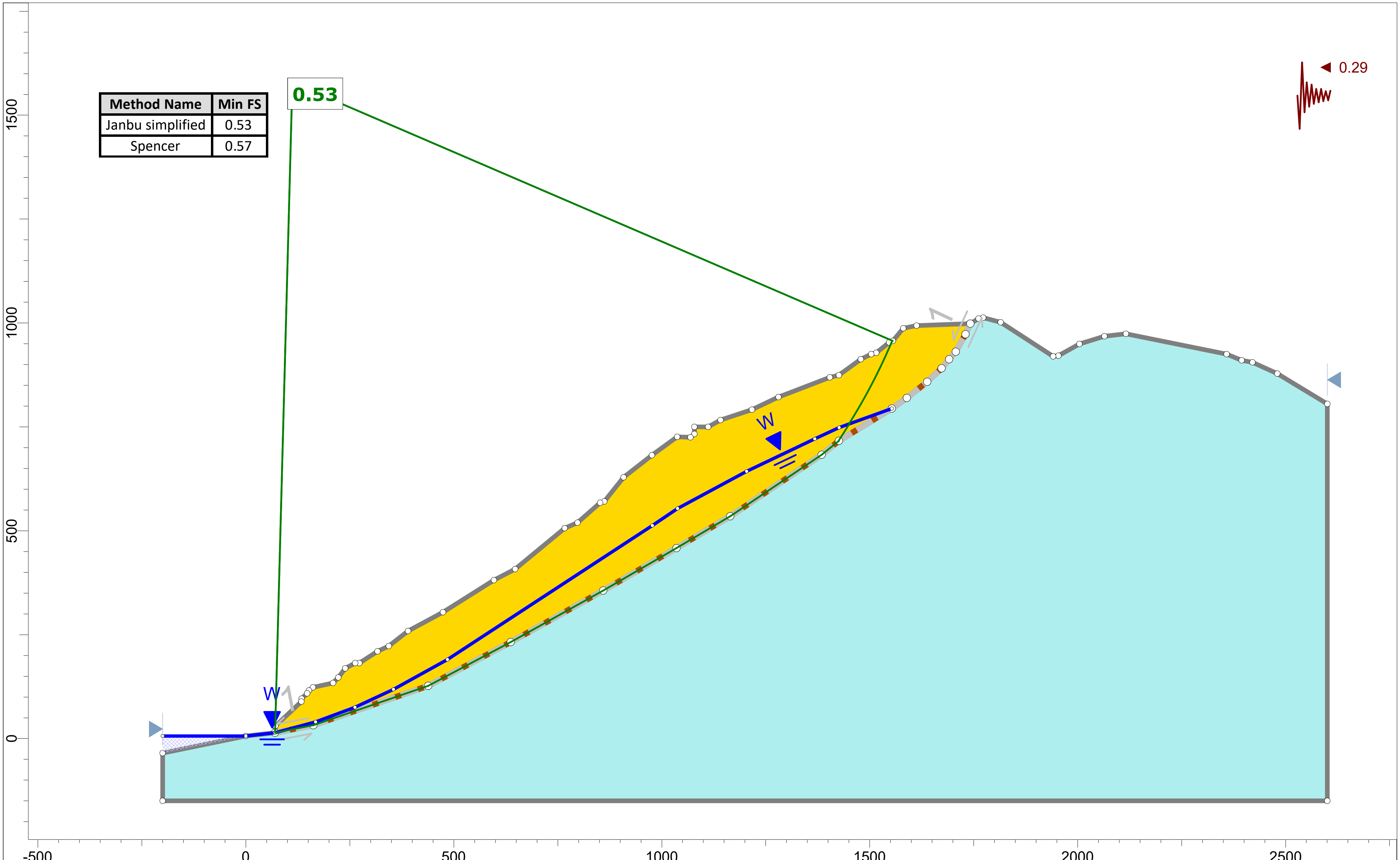

Method Name	Min FS
Janbu simplified	0.53
Spencer	0.57

KLEINFELDER
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Project No.:	TO C Section 3.slmd - Existing Condition - Wet, Pseudostatic	**Figure**
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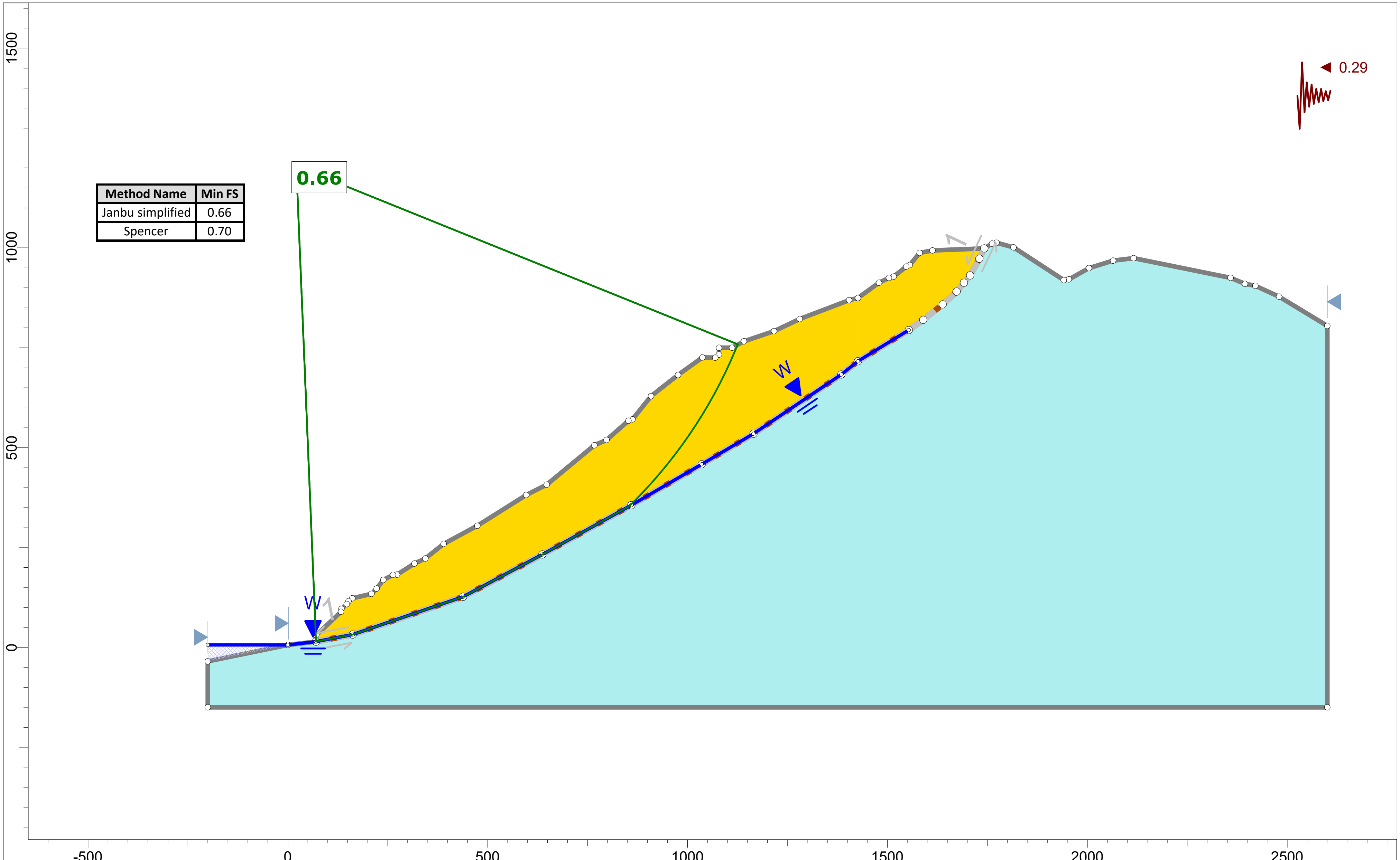
Date: 11/9/2021		



Method Name	Min FS
Janbu simplified	0.66
Spencer	0.70

KLEINFELDER
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Project No.:	TO C Section 3.slmd - Existing Condition - Dewatered, Pseudostatic	**Figure**
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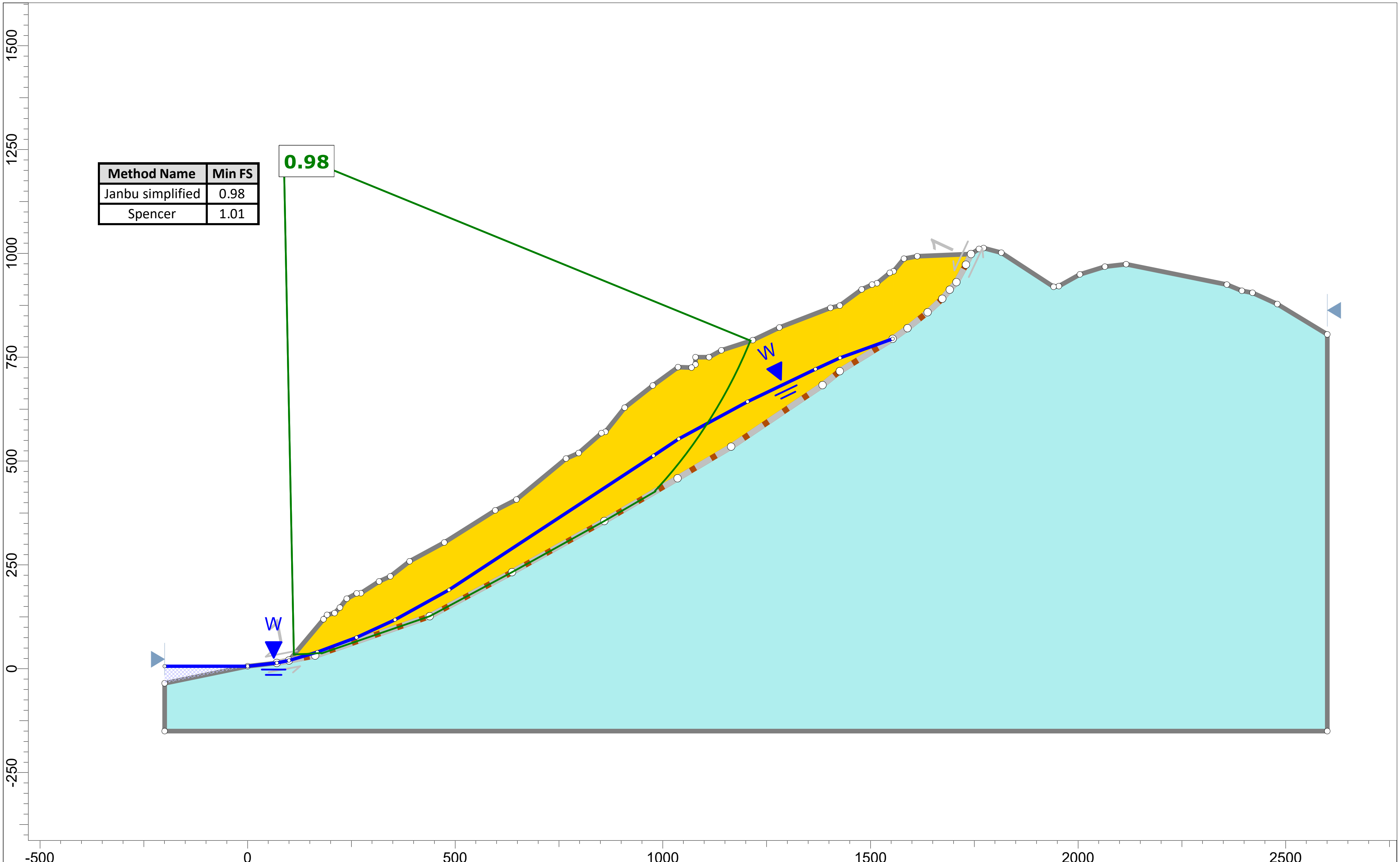
Checked by:	**Last Chance Grade**	
Date: 11/9/2021		



KLEINFELDER
Bright People. Right Solutions.

Project No.:	TO C Section 3.slmd - Coastal Erosion - Wet, Static	**Figure**
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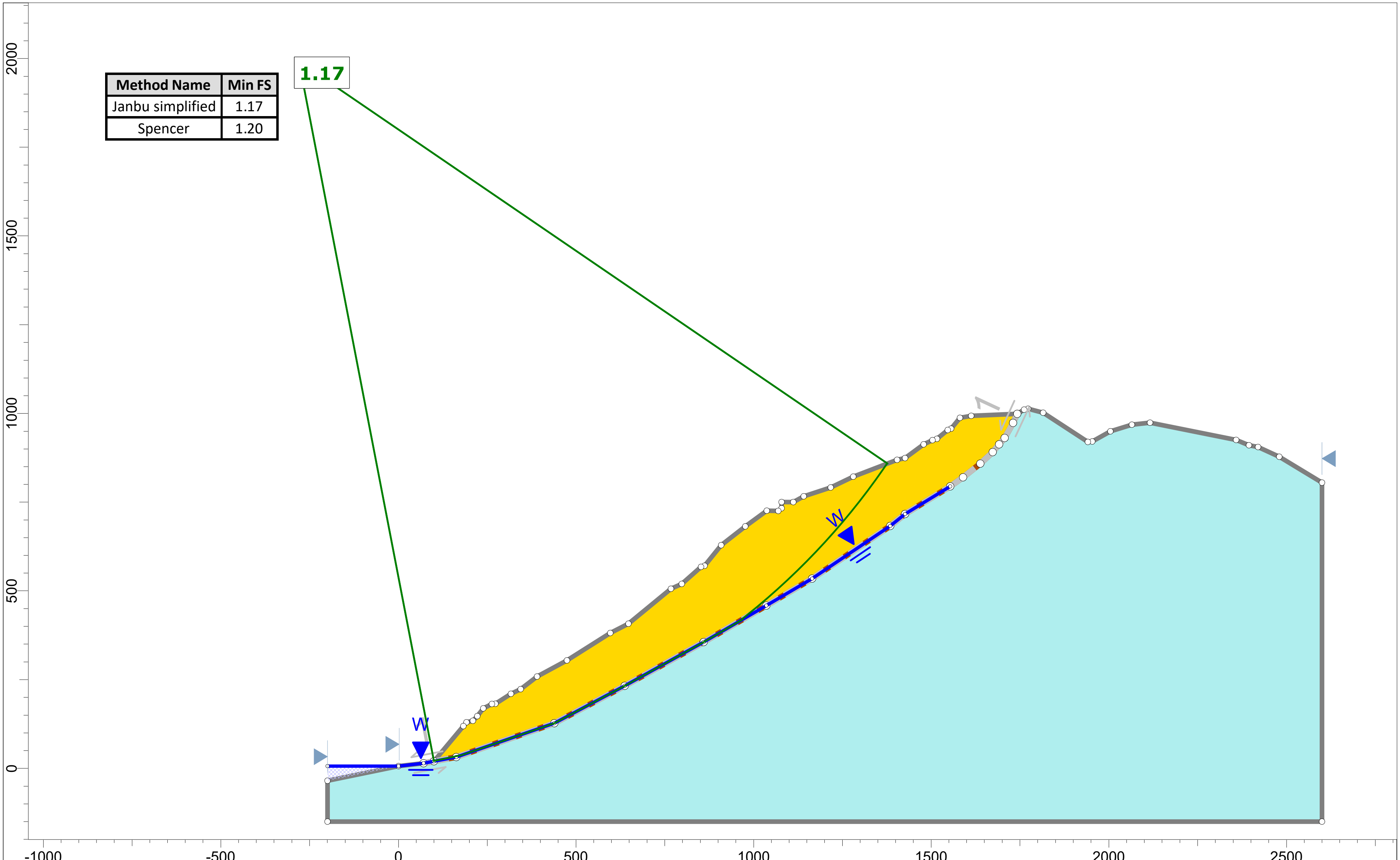
Date:11/9/2021		
Checked by:	**Last Chance Grade**	
Date: 11/9/2021		



KLEINFELDER
Bright People. Right Solutions.

Project No.:	TO C Section 3.slmd - Coastal Erosion - Dewatered, Static	**Figure**
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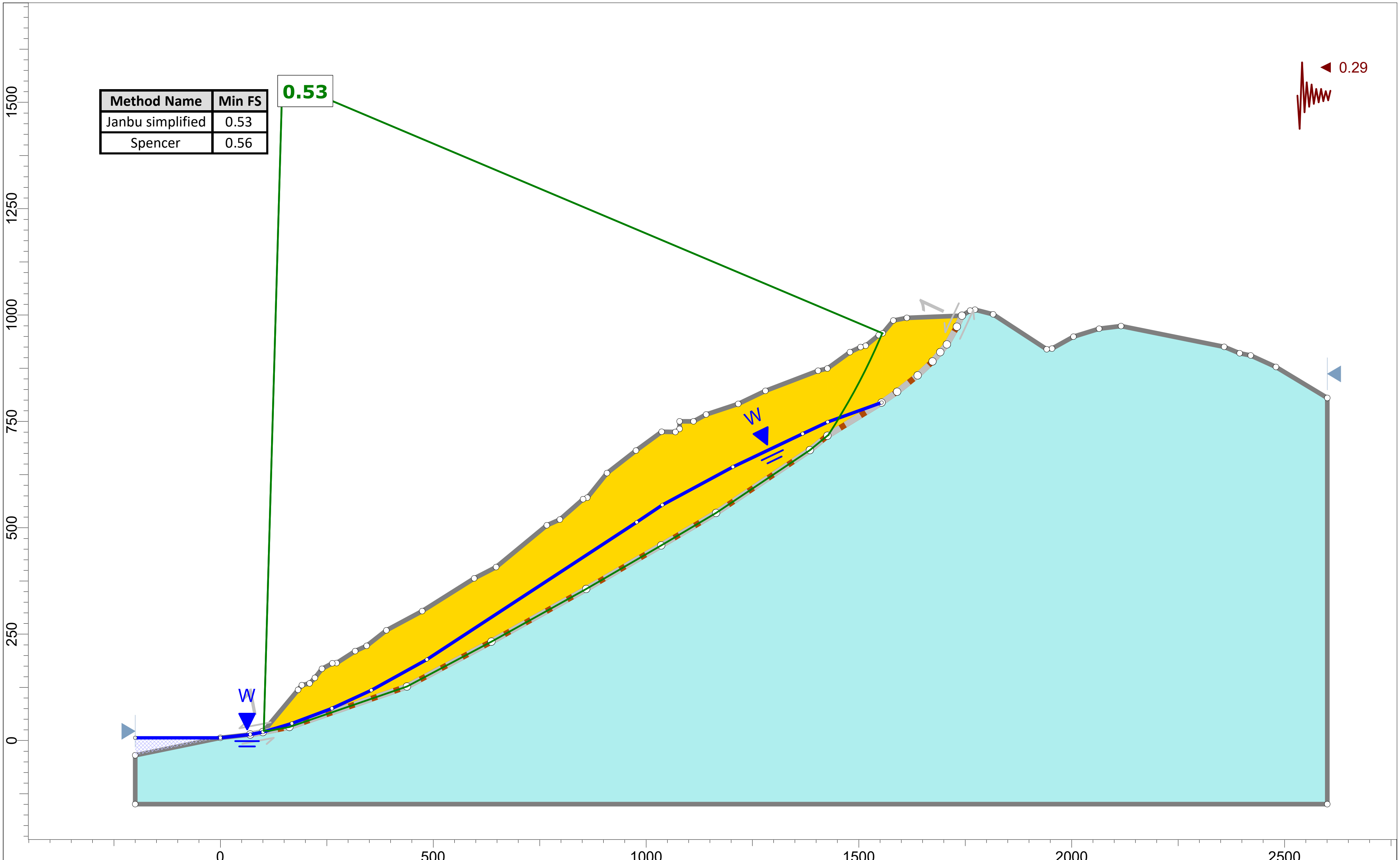
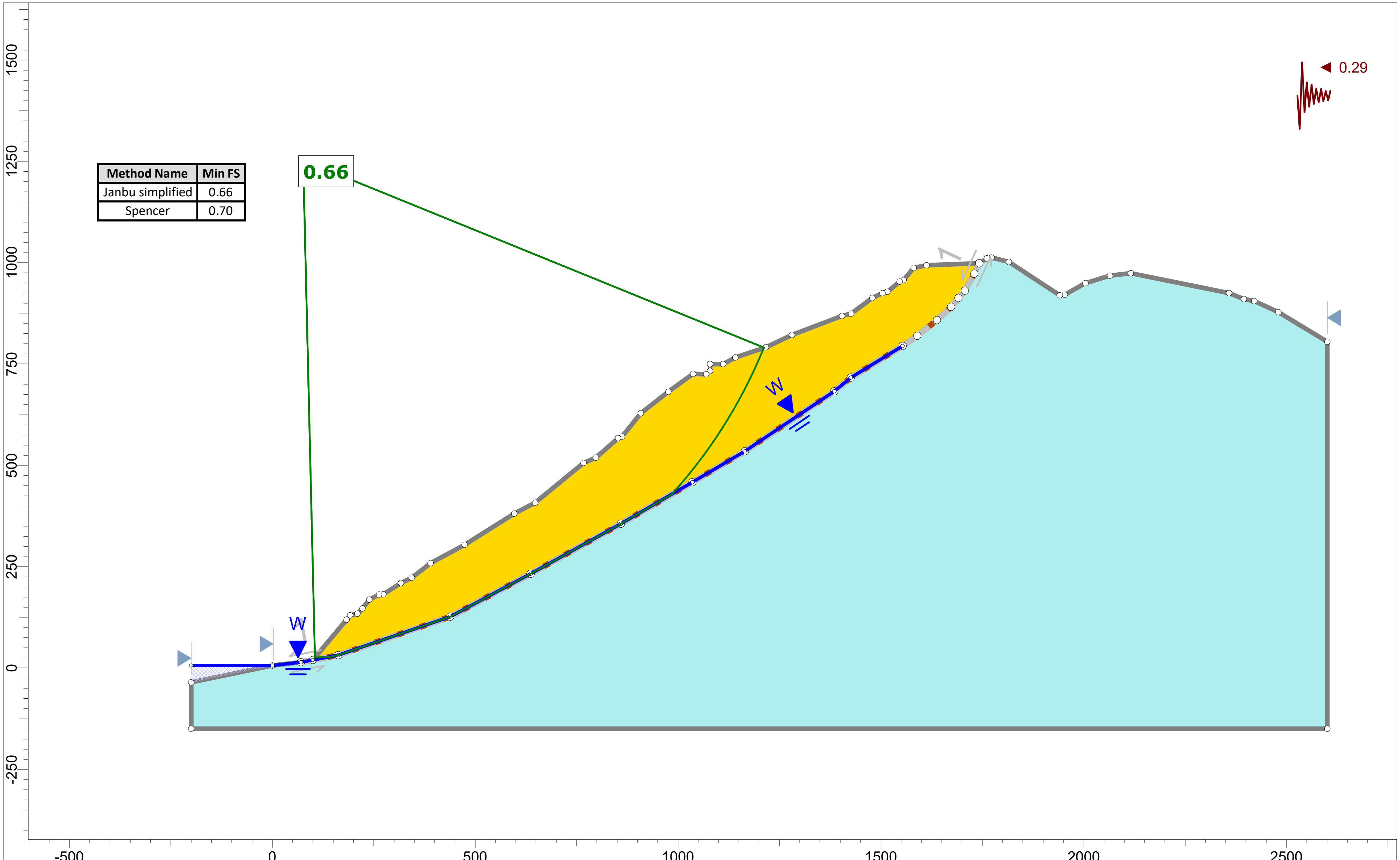
Date:11/9/2021		
Checked by:	**Last Chance Grade**	
Date: 11/9/2021		



KLEINFELDER
Bright People. Right Solutions.

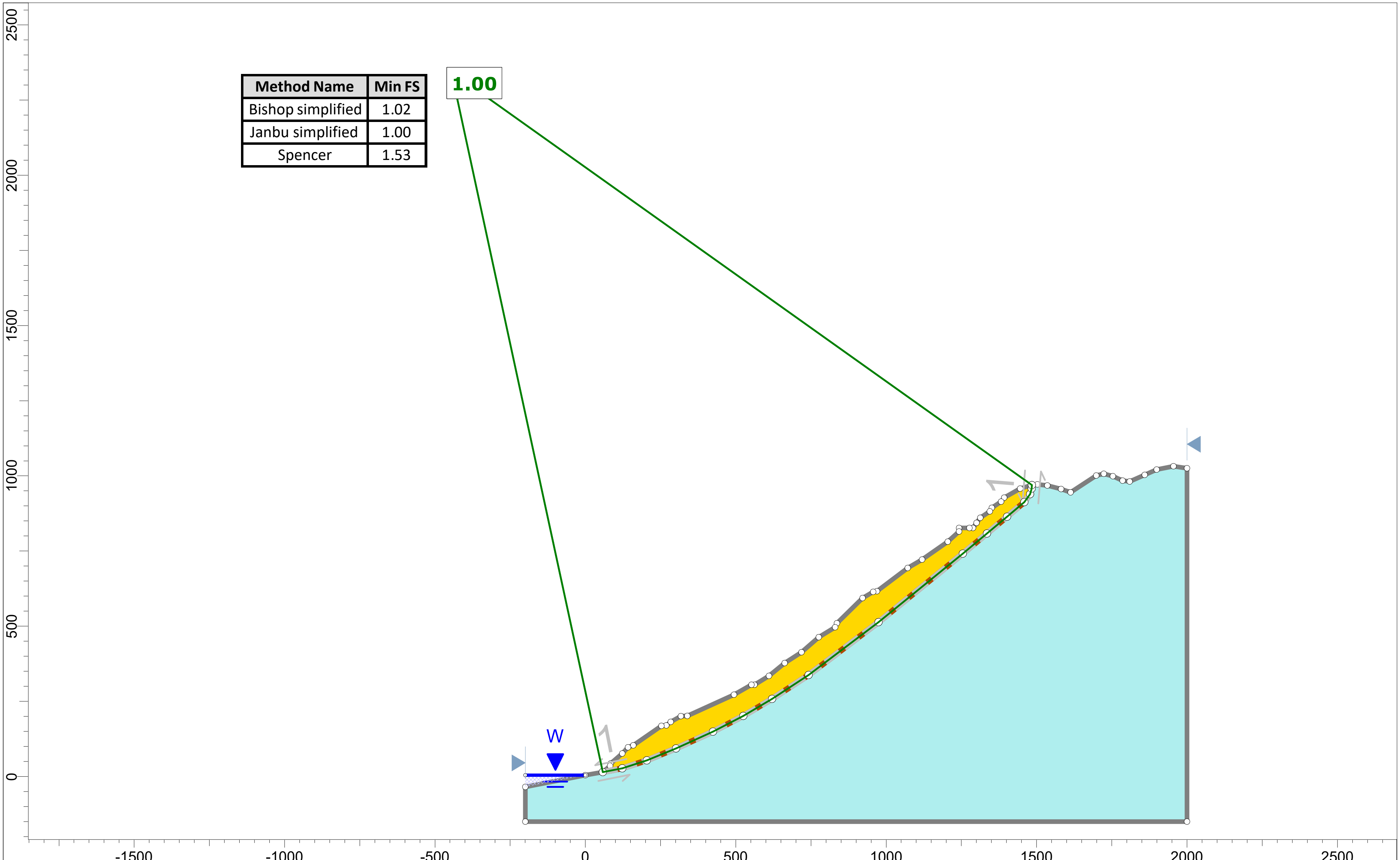
Project No.:	TO C Section 3.slmd - Coastal Erosion - Wet, Pseudostatic		**Figure**
Drawn by: Renie Yuen			
Date:11/9/2021			**A-3-7**
Checked by:	**Last Chance Grade**		
Date: 11/9/2021			



KLEINFELDER
Bright People. Right Solutions.

Project No.:	TO C Section 3.slmd - Coastal Erosion - Dewatered, Pseudostatic	**Figure**
Drawn by: Renie Yuen		
Date:11/9/2021		**A-3-8**
Checked by:	**Last Chance Grade**	
Date: 11/9/2021		

APPENDIX A4 Section 4 North LCG Complex



Method Name	Min FS
Bishop simplified	1.02
Janbu simplified	1.00
Spencer	1.53

KLEINFELDER
Bright People. Right Solutions.

Project No.:	TO C Section 4.slmd - Existing Condition, Static	Figure
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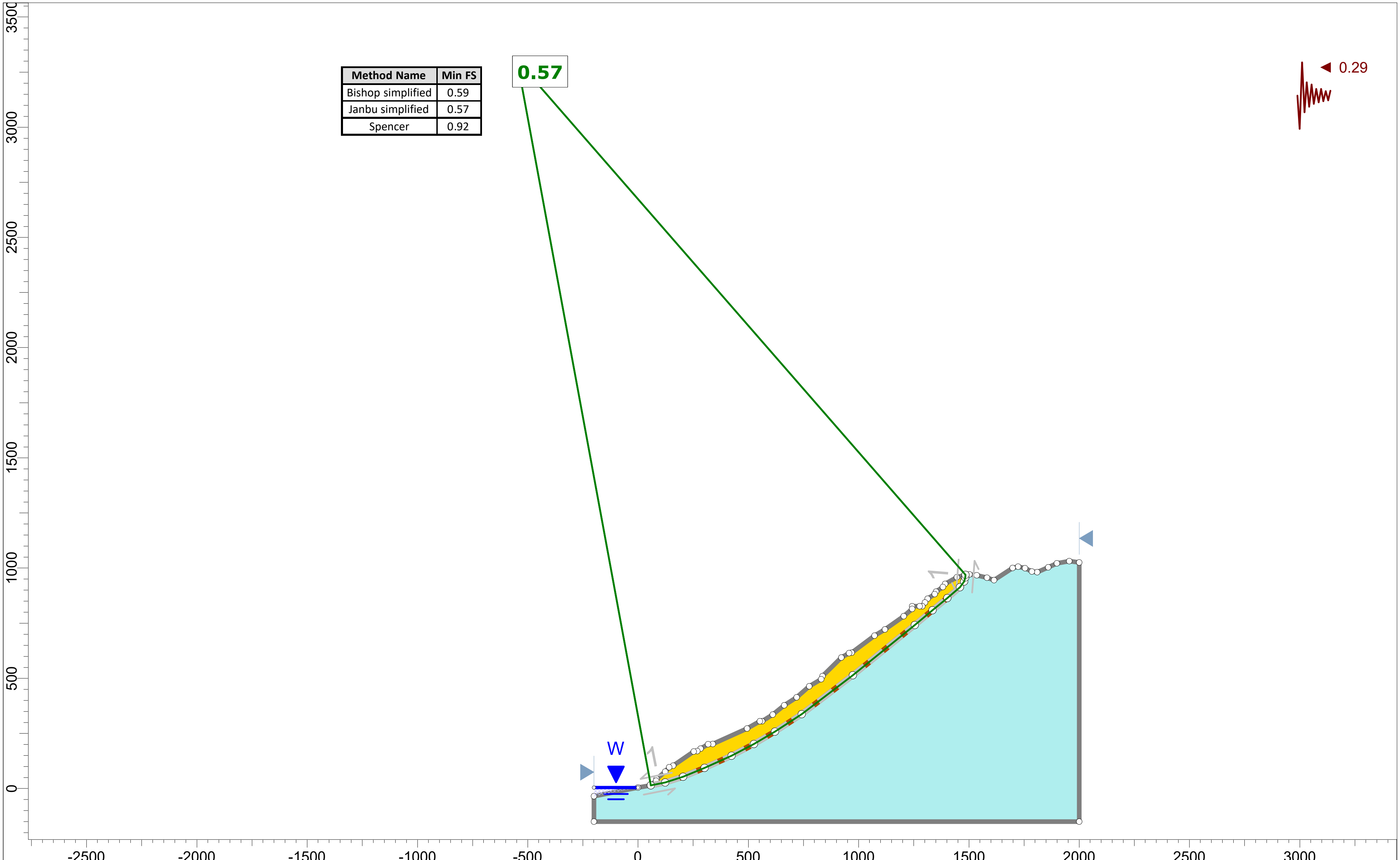
Date:11/9/2021		
Checked by:	**Last Chance Grade**	
Date: 11/9/2021		



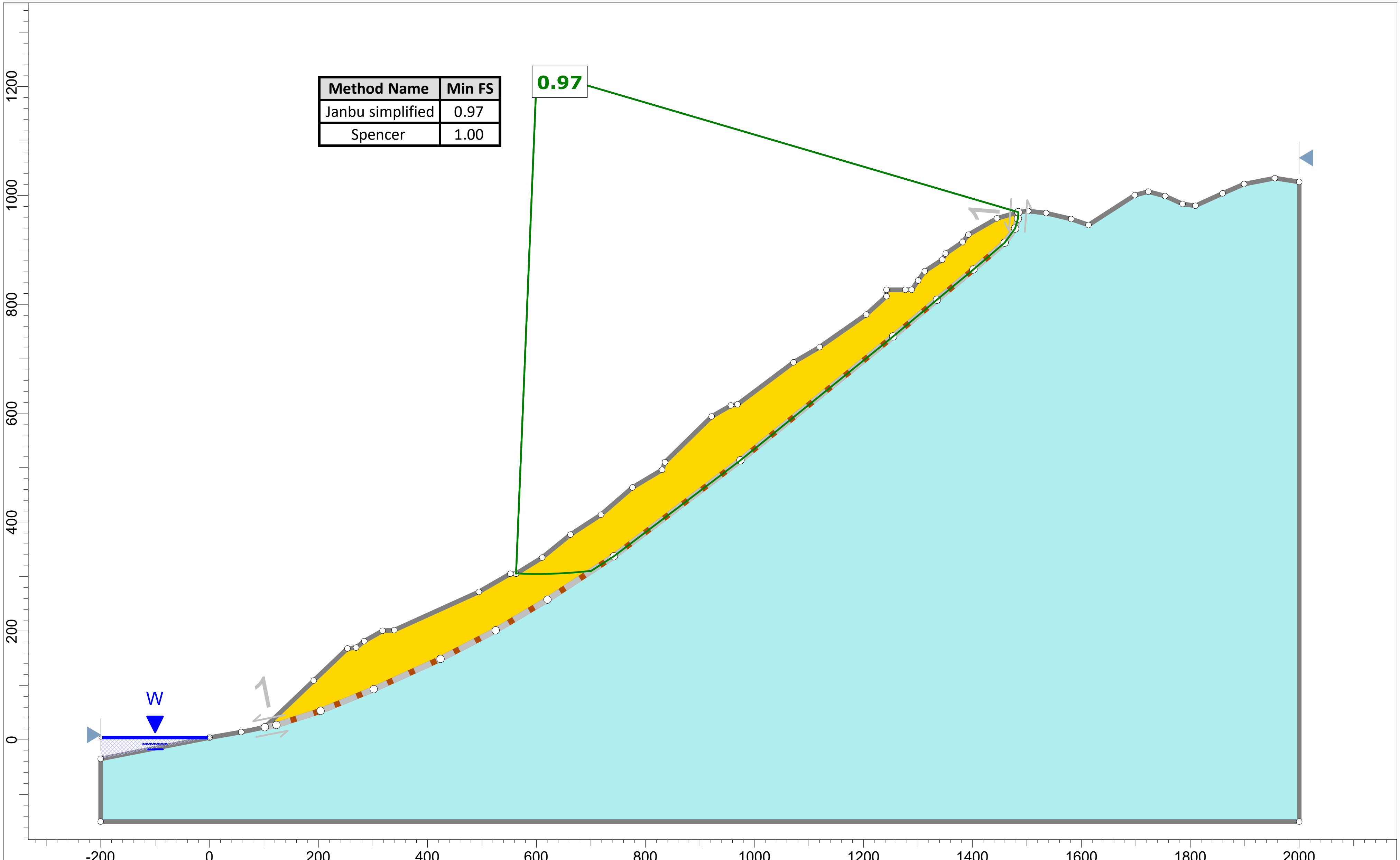
Method Name	Min FS
Bishop simplified	0.59
Janbu simplified	0.57
Spencer	0.92

KLEINFELDER
Bright People. Right Solutions.

Project No.:	
Drawn by: Renie Yuen	TO C Section 4.slmd - Existing Condition, Pseudostatic
Date:11/9/2021	
Checked by:	Last Chance Grade
Date: 11/9/2021	

Figure

A-4-2

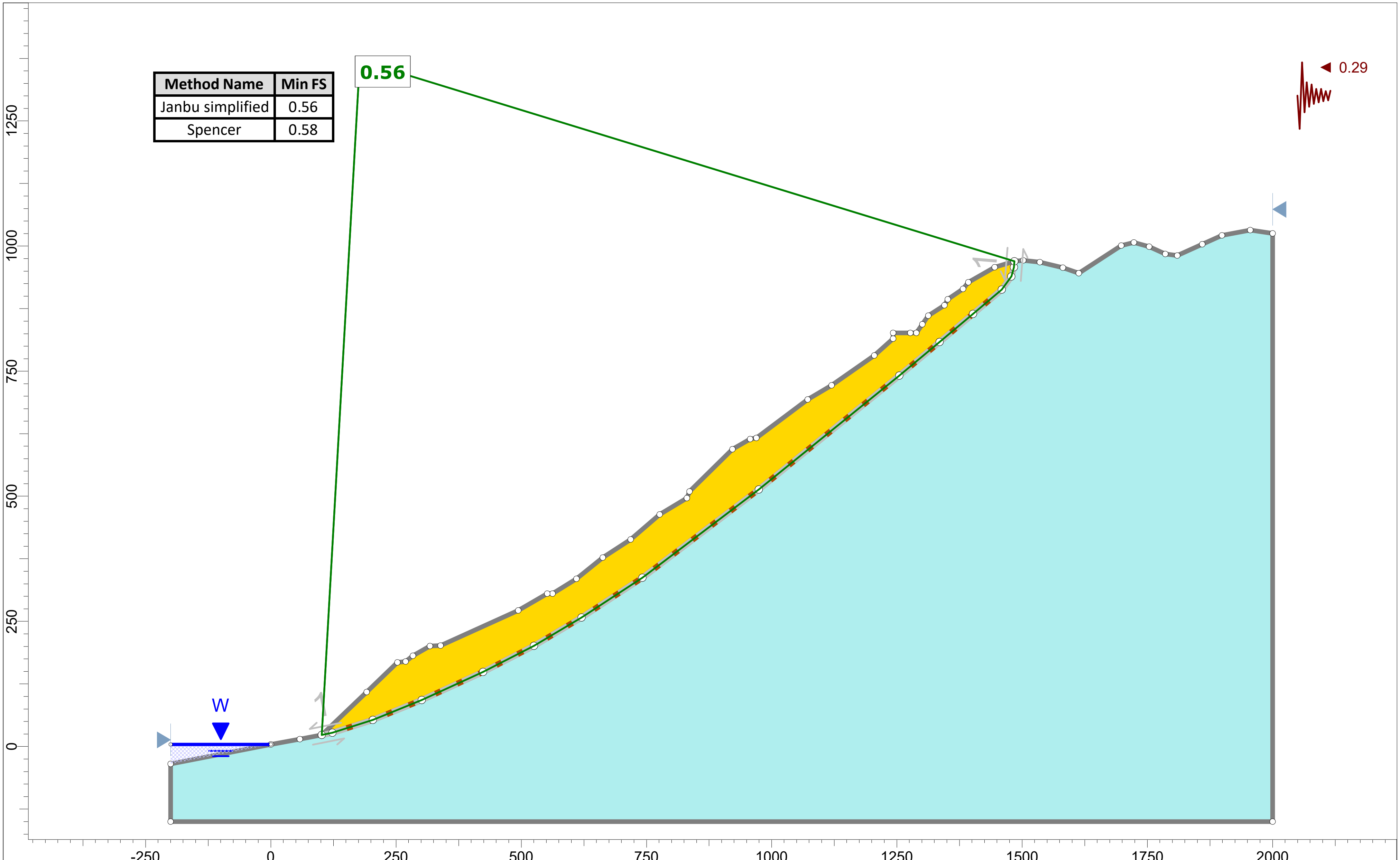


KLEINFELDER
Bright People. Right Solutions.

Project No.:	TO C Section 4.slmd - Coastal Erosion - Wet, Static
Drawn by: Renie Yuen	
Date:11/9/2021	
Checked by:	Last Chance Grade
Date: 11/9/2021	

Figure

A-4-3



Project No.:	TO C Section 4.slmd - Coastal Erosion - Wet, Pseudostatic	**Figure**
Drawn by: Renie Yuen		**A-4-4**
Date:11/9/2021		
Checked by:	**Last Chance Grade**	
Date: 11/9/2021		

KLEINFELDER
Bright People. Right Solutions.

APPENDIX A5 Slope Stability Analyses Summary

Case			Section 3	Section 5	Section 6	Section 7
			Factor of Safety (Circular, Janbu simplified/Spencer)	**Factor of Safety (Circular, Janbu simplified/Spencer)**	**Factor of Safety (Circular, Janbu simplified/Spencer)**	**Factor of Safety (Circular, Janbu simplified/Spencer)**
			Assumed discontinuous rupture surface (unconstrained search limits). Earth Flow Landslide: c=250 psf, phi=26 deg Melange: c=500 psf, phi=28 deg	*Groundwater not modeled in Broken Formation. Rock/Debris Landslide: c=1500 psf, phi=36 deg Broken Formation: c=1500 psf, phi=40 deg*	*Groundwater not modeled in Broken Formation. Rock/Debris Landslide: c=1500 psf, phi=36 deg Broken Formation: c=1500 psf, phi=40 deg*	*Groundwater not present in Rock/Debris Landslide Rock/Debris Landslide: c=1500 psf, phi=36 deg Broken Formation: c=1500 psf, phi=40 deg*
Existing Condition	Steady-State	Wet	0.89 / 0.92	0.99 / 1.01	0.98 / 1.01	1.00 / 1.53 / 1.02 (Bishop simplified)
			-critical slip surface at toe, does not cross rupture surface	-critical slip surface along rupture surface, majority of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	-critical slip surface along rupture surface, lower 2/3 of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	-critical slip surface along rupture surface, full slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium
		Dewatered	1.05 / 1.10	1.07 / 1.09	0.98 / 1.01	
			-critical slip surface at toe, does not cross rupture surface	-critical slip surface along rupture surface, majority of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	-critical slip surface along rupture surface, lower 2/3 of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	
	Pseudostatic (kh=0.29)	Wet	0.55 / 0.58	0.55 / 0.57	0.53 / 0.57	0.57 / 0.92 / 0.59 (Bishop simplified)
			-critical slip surface at lower 1/2 of slope, exits below toe, crosses rupture surface	-critical slip surface along rupture surface, majority of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	-critical slip surface along rupture surface, lower 4/5 of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	-critical slip surface along rupture surface, full slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium
		Dewatered	0.61 / 0.68	0.60 / 0.62	0.53 / 0.56	
			-critical slip surface at lower 1/5 of slope, exits below toe, crosses rupture surface	-critical slip surface along rupture surface, majority of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	-critical slip surface along rupture surface, lower 4/5 of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	
Coastal Erosion	Steady-State	Wet	0.95 / 1.00	0.98 / 1.00	1.14 / 1.19	0.97 / 1.00
			-critical slip surface at toe, does not cross rupture surface	-critical slip surface along rupture surface, lower 2/3 of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	-critical slip surface along rupture surface, lower 2/3 of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	-critical slip surface along rupture surface, upper 2/3 slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium
		Dewatered	0.98 / 1.03	1.04 / 1.08	1.17 / 1.20	
			-critical slip surface at toe, does not cross rupture surface	-critical slip surface along rupture surface, lower 2/3 of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	-critical slip surface along rupture surface, lower 3/4 of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	
	Pseudostatic (kh=0.29)	Wet	0.54 / 0.58	0.54 / 0.56	0.66 / 0.70	0.56 / 0.58
			-critical slip surface at lower 1/2 of slope, exits below toe, crosses rupture surface	-critical slip surface along rupture surface, lower 2/3 of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	-critical slip surface along rupture surface, lower 2/3 of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	-critical slip surface along rupture surface, full slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium
		Dewatered	0.59 / 0.64	0.60 / 0.63	0.66 / 0.70	
			-critical slip surface at lower 1/5 of slope, exits below toe, crosses rupture surface	-critical slip surface along rupture surface, lower 2/3 of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	-critical slip surface along rupture surface, lower 2/3 of slope -colluvium at surface modeled as landslide deposits to force critical surface deeper than depth of colluvium	

APPENDIX B Preliminary Seismic Parameters

APPENDIX B1 Vs30 Calculations

Determination of V_{s30}

per Caltrans Methodology for Developing Design Response Spectrum for Seismic Design Recommendations (2012)

Project: **Last Chance Grade**
Project No. 20-103
By: PSS Date: 10/11/2023
Check'd By: AZ Date: 10/20/2023

Borehole ID: **RC-18-001** **El. 344' approx.**
Borehole Depth: 85.25 ft
Depth to Grundwater: 14.00 ft
ER_i 60 %

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
5	Sand	S	4	120.0	4	4	29	144	7.5	0.052045
10	Rock	S	100	120.0	100	100	57	474	5.0	0.010557
15	Sand	S	4	120.0	4	4	83	185	5.0	0.026996
20	Sand	S	12	120.0	12	12	97	213	5.0	0.023431
25	Rock	S	33	120.0	33	33	111	333	5.0	0.015036
30	Sand	S	15	120.0	15	15	125	231	5.0	0.021619
35	Rock	S	45	120.0	45	45	138	367	5.0	0.013620
40	Rock	S	48	120.0	48	48	152	375	5.0	0.013342
45	Rock	S	60	120.0	60	60	166	402	5.0	0.012425
50	Rock	S	42	120.0	42	42	180	359	5.0	0.013923
55	Rock	S	32	120.0	32	32	194	329	5.0	0.015185
60	Sand	S	24	120.0	24	24	207	273	5.0	0.018324
65	Rock	S	37	120.0	37	37	221	345	5.0	0.014497
70	Rock	S	53	120.0	53	53	235	387	5.0	0.012927
75	Rock	S	46	120.0	46	46	249	370	5.0	0.013525
80	Rock	S	100	120.0	100	100	262	474	5.0	0.010557
85	Rock	S	100	120.0	100	100	276	474	2.8	0.005806
								Total:	85.25	0.29382

Calculation:
Total unit weight are based on laboratory tests and/or estimated based on N_{60} values
Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)
Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i/V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively — Vs(d) = 290

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec — V_{s30} = 308

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohessionless Soils

for "Sand", $V_s = \exp(4.045 + 0.096*\ln(N_{60}) + 0.236*\ln(\sigma_v')) <= 380$m/sec

for "Silt", $V_s = \exp(3.783 + 0.178*\ln(N_{60}) + 0.231*\ln(\sigma_v')) <= 380$m/sec

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60}=N*ER_i/60$)

Cohesive Soils ("Clay")

$V_s = \exp(3.996 + 0.230*\ln(N_{60}) + 0.164*\ln(\sigma_v')) <= 310$m/sec

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319} <= 560$m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}= 307.6 m/sec

Determination of V_{s30}
per Caltrans Methodology for Developing Design Acceleration Response Spectrum for Seismic Design Recommendations (2021)

Project: **Last Chance Grade**
Project No. 20-103
By: PSS Date: 10/11/2023
Check'd By: AZ Date: 10/20/2023

Borehole ID: **RC-18-001** **El. 344' approx.**
Borehole Depth: 85.25 ft
Depth to Grundwater: 14.00 ft
ER_i 60 %

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	ASF Age	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
5	Sand	S	4	120.0	Quarternary	4	4	29	89	7.5	0.083957
10	Rock	S	100	120.0		100	100	57	474	5.0	0.010557
15	Sand	S	4	120.0	Quarternary	4	4	83	114	5.0	0.043828
20	Sand	S	12	120.0	Quarternary	12	12	97	152	5.0	0.032862
25	Rock	S	33	120.0		33	33	111	333	5.0	0.015036
30	Sand	S	15	120.0	Quarternary	15	15	125	170	5.0	0.029472
35	Rock	S	45	120.0		45	45	138	367	5.0	0.013620
40	Rock	S	48	120.0		48	48	152	375	5.0	0.013342
45	Rock	S	60	120.0		60	60	166	402	5.0	0.012425
50	Rock	S	42	120.0		42	42	180	359	5.0	0.013923
55	Rock	S	32	120.0		32	32	194	329	5.0	0.015185
60	Sand	S	24	120.0	Quarternary	24	24	207	213	5.0	0.023528
65	Rock	S	37	120.0		37	37	221	345	5.0	0.014497
70	Rock	S	53	120.0		53	53	235	387	5.0	0.012927
75	Rock	S	46	120.0		46	46	249	370	5.0	0.013525
80	Rock	S	100	120.0		100	100	262	474	5.0	0.010557
85	Rock	S	100	120.0		100	100	276	474	2.8	0.005806
									Total:	85.25	0.36505

Calculation:

Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)
Average shear wave velocity
$V_{s(d)} = d/\{\Sigma D_i / V_{s,i}\}$
where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively — Vs(d) = 234

If soil column depth d<30m (100 ft):
V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec — V_{s30} = 248
where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value
Cohesive Soils ("Clay")
$V_s = 26*(ASF)*(N_{60})$^0.17*(σ_v')^0.32)
Where, ASF =1.0 for Quaternary or 0.88 for Holocene and 1.12 for Pleistocene.
where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60}=N*ER_i/60$)
Cohessionless Soils ("Sand" and "Silt")
For "Silt", The SPT N60 correlation recommended above for cohesive soil layers is also recommended for silt layers.
For "Sand", $V_s = 30*(ASF)*(N_{60})$^0.23*(σ_v')^0.23)
Where, ASF =1.0 for Quaternary or 0.9 for Holocene and 1.17 for Pleistocene.
For "Gravel"

Young Sedimentary Rock (Tertiary deposits) ("Rock")
$V_s = 109*(N_{60})^{0.319} <= 560$ m/sec
Competent Rock
V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}= 247.6 m/sec

Determination of V_{s30}

per Caltrans Methodology for Developing Design Response Spectrum for Seismic Design Recommendations (2012)

Project: **Last Chance Grade**
Project No. 20-103
By: PSS   Date: 10/11/2023
Check'd By: AZ   Date: 10/20/2023

Borehole ID: **RC-19-001**   **El. 550' approx.**
Borehole Depth: 100.00 ft
Depth to Grundwater: 166.00 ft
ER_i 60 %

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
5	Rock	S	26	120.0	26	26	29	308	5.8	0.018658
6.5	Rock	S	18	120.0	18	18	37	274	2.5	0.009122
10	Rock	S	21	120.0	21	21	57	288	2.5	0.008684
11.5	Rock	S	51	120.0	51	51	66	382	2.5	0.006543
15	Rock	S	18	120.0	18	18	86	274	2.5	0.009122
16.5	Rock	S	23	120.0	23	23	95	296	2.5	0.008436
20	Rock	S	44	120.0	44	44	115	364	2.5	0.006859
21.5	Rock	S	19	120.0	19	19	124	279	79.3	0.284217
								Total:	100.00	0.35164

Calculation:

Total unit weight are based on laboratory tests and/or estimated based on N_{60} values

Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)

Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i / V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively   Vs(d) = 284

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec V_{s30} = 284

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohessionless Soils

for "Sand", $V_s = \exp(4.045 + 0.096*\ln(N_{60}) + 0.236*\ln(\sigma_v')) <= 380$m/sec

for "Silt", $V_s = \exp(3.783 + 0.178*\ln(N_{60}) + 0.231*\ln(\sigma_v')) <= 380$m/sec

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60} = N*ER_i/60$)

Cohesive Soils ("Clay")

$V_s = \exp(3.996 + 0.230*\ln(N_{60}) + 0.164*\ln(\sigma_v')) <= 310$m/sec

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319} <= 560$m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}= 284.4 m/sec

Determination of V_{s30}

per Caltrans Methodology for Developing Design Response Spectrum for Seismic Design Recommendations (2012)

Project:	**Last Chance Grade**		
Project No.	20-103		
By:	PSS	Date:	10/11/2023
Check'd By:	AZ	Date:	10/20/2023

Borehole ID:	**RC-19-003**	**El. 860' approx.**
Borehole Depth:	100.00	ft
Depth to Grundwater:	10.00	ft
ER_i	60	%

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
7	Silt	S	100	120.0	100	100	40	234	7.8	0.033093
8.5	Silt	R	5	120.0	4	4	49	135	4.0	0.029660
15	Rock	S	50	120.0	50	50	71	380	5.8	0.015145
20	Rock	S	100	120.0	100	100	85	474	5.0	0.010557
25	Rock	S	100	120.0	100	100	99	474	5.0	0.010557
30	Rock	S	100	120.0	100	100	113	474	72.5	0.153078
								Total:	100.00	0.25209

Calculation:

Total unit weight are based on laboratory tests and/or estimated based on N_{60} values

Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)

Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i / V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively

Vs(d) = 397

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec

V_{s30} = 397

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohessionless Soils

for "Sand", $V_s = \exp(4.045 + 0.096*\ln(N_{60}) + 0.236*\ln(\sigma_v')) <= 380$ m/sec

for "Silt", $V_s = \exp(3.783 + 0.178*\ln(N_{60}) + 0.231*\ln(\sigma_v')) <= 380$ m/sec

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60}=N*ER_i/60$)

Cohesive Soils ("Clay")

$V_s = \exp(3.996 + 0.230*\ln(N_{60}) + 0.164*\ln(\sigma_v')) <= 310$ m/sec

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319} <= 560$ m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}= 396.7 m/sec

Determination of V_{s30}

per Caltrans Methodology for Developing Design Response Spectrum for Seismic Design Recommendations (2012)

Project:	**Last Chance Grade**		
Project No.	20-103		
By:	PSS	Date:	10/11/2023
Check'd By:	AZ	Date:	10/20/2023

Borehole ID:	**RC-20-005**	**El. 845' approx.**
Borehole Depth:	250.00	ft
Depth to Grundwater:	155.00	ft
ER_i	60	%

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
3	Silt	S	2	120.0	2	2	17	96	5.5	0.057309
8	Clay	S	2	120.0	2	2	46	119	3.5	0.029292
10	Rock	S	100	120.0	100	100	57	474	3.5	0.007390
15	Rock	S	34	120.0	34	34	86	336	5.0	0.014894
20	Rock	S	100	120.0	100	100	115	474	4.5	0.009501
24	Rock	S	100	120.0	100	100	138	474	5.0	0.010557
30	Rock	S	100	120.0	100	100	172	474	73.0	0.154133
								Total:	100.00	0.28308

Calculation:

Total unit weight are based on laboratory tests and/or estimated based on N_{60} values

Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)

Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i / V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively — Vs(d) = 353

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec — V_{s30} = 353

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohessionless Soils

for "Sand", $V_s = \exp(4.045 + 0.096*\ln(N_{60}) + 0.236*\ln(\sigma_v')) <= 380$m/sec

for "Silt", $V_s = \exp(3.783 + 0.178*\ln(N_{60}) + 0.231*\ln(\sigma_v')) <= 380$m/sec

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60}=N*ER_i/60$)

Cohesive Soils ("Clay")

$V_s = \exp(3.996 + 0.230*\ln(N_{60}) + 0.164*\ln(\sigma_v')) <= 310$m/sec

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319} <= 560$m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}= 353.3 m/sec

Determination of V_{s30}

per Caltrans Methodology for Developing Design Response Spectrum for Seismic Design Recommendations (2012)

Project: **Last Chance Grade**
Project No. 20-103
By: PSS Date: 10/11/2023
Check'd By: AZ Date: 10/20/2023

Borehole ID: **RC-20-006** **El. 625' approx.**
Borehole Depth: 65.00 ft
Depth to Grundwater: 60.00 ft
ER_i 60 %

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
5	Clay	S	11	120.0	11	11	29	164	7.5	0.045808
10	Clay	S	4	120.0	4	4	57	145	5.0	0.034397
15	Clay	S	8	120.0	8	8	86	182	5.0	0.027442
20	Sand	S	7	120.0	7	7	115	211	5.0	0.023707
25	Sand	S	6	120.0	6	6	144	219	5.0	0.022826
30	Sand	S	11	120.0	11	11	172	242	5.0	0.020629
35	Sand	S	10	120.0	10	10	201	249	5.0	0.020075
40	Sand	S	18	120.0	18	18	230	272	5.0	0.018385
45	Sand	S	26	120.0	26	26	259	290	5.0	0.017260
50	Rock	S	100	120.0	100	100	287	474	8.0	0.016891
61	Sand	S	29	120.0	29	29	347	314	9.5	0.030266
								Total:	65.00	0.27768

Calculation:
Total unit weight are based on laboratory tests and/or estimated based on N_{60} values
Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)
Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i / V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively Vs(d) = 234

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec V_{s30} = 270

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohessionless Soils

for "Sand", $V_s = \exp(4.045 + 0.096*\ln(N_{60}) + 0.236*\ln(\sigma_v')) <= 380$ m/sec

for "Silt", $V_s = \exp(3.783 + 0.178*\ln(N_{60}) + 0.231*\ln(\sigma_v')) <= 380$ m/sec

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60}=N*ER_i/60$)

Cohesive Soils ("Clay")

$V_s = \exp(3.996 + 0.230*\ln(N_{60}) + 0.164*\ln(\sigma_v')) <= 310$ m/sec

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319} <= 560$ m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}= 269.9 m/sec

Determination of V_{s30}
per Caltrans Methodology for Developing Design Response Spectrum for Seismic Design Recommendations (2012)

Project: **Last Chance Grade**
Project No. 20-103
By: PSS Date: 10/11/2023
Check'd By: AZ Date: 10/20/2023

Borehole ID: **RC-20-006** **El. 625' approx.**
Borehole Depth: 65.00 ft
Depth to Grundwater: 60.00 ft
ER_i 60 %

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	ASF Age	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
5	Clay	S	11	120.0	Quarternary	11	11	29	114	7.5	0.065523
10	Clay	S	4	120.0	Quarternary	4	4	57	120	5.0	0.041558
15	Clay	S	8	120.0	Quarternary	8	8	86	154	5.0	0.032444
20	Sand	S	7	120.0	Quarternary	7	7	115	140	5.0	0.035776
25	Sand	S	6	120.0	Quarternary	6	6	144	142	5.0	0.035213
30	Sand	S	11	120.0	Quarternary	11	11	172	170	5.0	0.029373
35	Sand	S	10	120.0	Quarternary	10	10	201	173	5.0	0.028978
40	Sand	S	18	120.0	Quarternary	18	18	230	204	5.0	0.024548
45	Sand	S	26	120.0	Quarternary	26	26	259	228	5.0	0.021954
50	Rock	S	100	120.0		100	100	287	474	8.0	0.016891
61	Sand	S	29	120.0	Quarternary	29	29	347	250	9.5	0.038004
									Total:	65.00	0.37026

Calculation:
Total unit weight are based on laboratory tests and/or estimated based on N_{60} values
Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)
Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i/V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively — Vs(d) = 176

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = (1.45-0.015*d)*$V_{s(d)}$ in unit of m/sec — V_{s30} = 202

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohesive Soils ("Clay")

V_s = 26*(ASF)*(N_{60})^0.17*(σ_v')^0.32)

Where, ASF =1.0 for Quaternary or 0.88 for Holocene and 1.12 for Pleistocene.

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., N_{60}=N*ER_i/60)

Cohessionless Soils ("Sand" and "Silt")

For "Silt", The SPT N60 correlation recommended above for cohesive soil layers is also recommended for silt layers.

For "Sand", V_s = 30*(ASF)*(N_{60})^0.23*(σ_v')^0.23)

Where, ASF =1.0 for Quaternary or 0.9 for Holocene and 1.17 for Pleistocene.

For "Gravel" V_s = 53*(N_{60})^0.19*(σ_v')^0.18) for Holocene

V_s = 115*(N_{60})^0.17*(σ_v')^0.12) for Pleistocene

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319}$ <= 560m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}=	**202.4**	**m/sec**

Determination of V_{s30}

per Caltrans Methodology for Developing Design Response Spectrum for Seismic Design Recommendations (2012)

Project: **Last Chance Grade**
Project No. 20-103
By: PSS Date: 10/11/2023
Check'd By: AZ Date: 10/20/2023

Borehole ID: **RC-20-011** **El. 698' approx.**
Borehole Depth: 302.60 ft
Depth to Grundwater: 144.00 ft
ER_i 60 %

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
2	Sand	S	2	120.0	2	2	11	109	4.5	0.041433
7	Silt	S	2	120.0	2	2	40	117	5.0	0.042838
12	Silt	S	5	120.0	5	5	69	156	5.0	0.032131
17	Sand	S	15	120.0	15	15	98	218	4.0	0.018317
20	Sand	S	27	120.0	27	27	115	240	3.5	0.014578
24	Rock	S	100	120.0	100	100	138	474	78.0	0.164690
								Total:	100.00	0.31399

Calculation:
Total unit weight are based on laboratory tests and/or estimated based on N_{60} values
Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)
Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i / V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively Vs(d) = 318

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec V_{s30} = 318

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohessionless Soils

for "Sand", $V_s = \exp(4.045 + 0.096*\ln(N_{60}) + 0.236*\ln(\sigma_v')) <= 380$m/sec

for "Silt", $V_s = \exp(3.783 + 0.178*\ln(N_{60}) + 0.231*\ln(\sigma_v')) <= 380$m/sec

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60}=N*ER_i/60$)

Cohesive Soils ("Clay")

$V_s = \exp(3.996 + 0.230*\ln(N_{60}) + 0.164*\ln(\sigma_v')) <= 310$m/sec

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319} <= 560$m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}= 318.5 m/sec

Determination of V_{s30}

per Caltrans Methodology for Developing Design Response Spectrum for Seismic Design Recommendations (2012)

Project: **Last Chance Grade**
Project No. 20-103
By: PSS   Date: 10/11/2023
Check'd By: AZ   Date: 10/20/2023

Borehole ID: **RC-20-013**   **El. 726' approx.**
Borehole Depth: 134.70 ft
Depth to Grundwater: 133.00 ft
ER_i 60 %

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
6	Sand	S	54	120.0	54	54	34	193	8.5	0.044009
11	Sand	S	30	120.0	30	30	63	211	5.0	0.023740
16	Sand	S	19	120.0	19	19	92	220	30.0	0.136227
71	Rock	S	100	120.0	100	100	408	474	56.5	0.119295
								Total:	100.00	0.32327

Calculation:
Total unit weight are based on laboratory tests and/or estimated based on N_{60} values
Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)
Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i / V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively   Vs(d) = 309

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec V_{s30} = 309

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohessionless Soils

for "Sand", $V_s = \exp(4.045 + 0.096*\ln(N_{60}) + 0.236*\ln(\sigma_v')) <= 380$ m/sec

for "Silt", $V_s = \exp(3.783 + 0.178*\ln(N_{60}) + 0.231*\ln(\sigma_v')) <= 380$ m/sec

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60} = N*ER_i/60$)

Cohesive Soils ("Clay")

$V_s = \exp(3.996 + 0.230*\ln(N_{60}) + 0.164*\ln(\sigma_v')) <= 310$ m/sec

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319} <= 560$ m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}= 309.3 m/sec

Determination of V_{s30}

per Caltrans Methodology for Developing Design Response Spectrum for Seismic Design Recommendations (2012)

Project:	**Last Chance Grade**		
Project No.	20-103		
By:	PSS	Date:	10/11/2023
Check'd By:	AZ	Date:	10/20/2023

Borehole ID:	**RC-20-014**	**El. 796' approx.**
Borehole Depth:	300.00	ft
Depth to Grundwater:	166.00	ft
ER_i	60	%

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
5	Clay	S	1	120.0	1	1	29	94	7.5	0.079517
10	Clay	S	5	120.0	5	5	57	153	5.0	0.032676
15	Clay	S	8	120.0	8	8	86	182	5.0	0.027442
20	Sand	S	19	120.0	19	19	115	232	5.0	0.021540
25	Sand	S	25	120.0	25	25	144	251	5.0	0.019903
30	Sand	S	27	120.0	27	27	172	264	5.0	0.018925
35	Sand	S	18	120.0	18	18	201	264	5.0	0.018973
40	Rock	S	100	120.0	100	100	230	474	62.5	0.131963
								Total:	100.00	0.35094

Calculation:

Total unit weight are based on laboratory tests and/or estimated based on N_{60} values

Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)

Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i / V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively — Vs(d) = 285

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec — V_{s30} = 285

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohessionless Soils

for "Sand", $V_s = \exp(4.045 + 0.096*\ln(N_{60}) + 0.236*\ln(\sigma_v')) <= 380$m/sec

for "Silt", $V_s = \exp(3.783 + 0.178*\ln(N_{60}) + 0.231*\ln(\sigma_v')) <= 380$m/sec

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60}=N*ER_i/60$)

Cohesive Soils ("Clay")

$V_s = \exp(3.996 + 0.230*\ln(N_{60}) + 0.164*\ln(\sigma_v')) <= 310$m/sec

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319} <= 560$m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}=	**284.9**	**m/sec**

Determination of V_{s30}

per Caltrans Methodology for Developing Design Acceleration Response Spectrum for Seismic Design Recommendations (2021)

Project:	**Last Chance Grade**		
Project No.	20-103		
By:	PSS	Date:	10/11/2023
Check'd By:	AZ	Date:	10/20/2023

Borehole ID:	**RC-20-014**	**El. 797' approx.**
Borehole Depth:	300.00	ft
Depth to Grundwater:	166.00	ft
ER_i	60	%

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	ASF Age	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
5	Clay	S	1	120.0	Quarternary	1	1	29	76	7.5	0.098498
10	Clay	S	5	120.0	Quarternary	5	5	57	125	5.0	0.040011
15	Clay	S	8	120.0	Quarternary	8	8	86	154	5.0	0.032444
20	Sand	S	19	120.0	Quarternary	19	19	115	176	5.0	0.028435
25	Sand	S	25	120.0	Quarternary	25	25	144	197	5.0	0.025360
30	Sand	S	27	120.0	Quarternary	27	27	172	209	5.0	0.023892
35	Sand	S	18	120.0	Quarternary	18	18	201	198	5.0	0.025314
40	Rock	S	100	120.0		100	100	230	474	62.5	0.131963
									Total:	100.00	0.40592

Calculation:

Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)

Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i / V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively — Vs(d) = 246

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec — V_{s30} = 246

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohesive Soils ("Clay")

$V_s = 26*(ASF)*(N_{60})\text{^}0.17*(\sigma_v')\text{^}0.32)$

Where, ASF =1.0 for Quaternary or 0.88 for Holocene and 1.12 for Pleistocene.

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60}=N*ER_i/60$)

Cohessionless Soils ("Sand" and "Silt")

For "Silt", The SPT N60 correlation recommended above for cohesive soil layers is also recommended for silt layers.

For "Sand", $V_s = 30*(ASF)*(N_{60})\text{^}0.23*(\sigma_v')\text{^}0.23)$

Where, ASF =1.0 for Quaternary or 0.9 for Holocene and 1.17 for Pleistocene.

For "Gravel" $V_s = 53*(N_{60})\text{^}0.19*(\sigma_v')\text{^}0.18)$ for Holocene

$V_s = 115*(N_{60})\text{^}0.17*(\sigma_v')\text{^}0.12)$ for Pleistocene

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319}$ <= 560m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}=	**246.4**	**m/sec**

Determination of V_{s30}

per Caltrans Methodology for Developing Design Response Spectrum for Seismic Design Recommendations (2012)

Project: **Last Chance Grade**
Project No. 20-103
By: PSS   Date: 10/11/2023
Check'd By: AZ   Date: 10/20/2023

Borehole ID: **RC-20-016**   **El. 634' approx.**
Borehole Depth: 300.00 ft
Depth to Grundwater: 136.00 ft
ER_i 60 %

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
2	Sand	S	29	120.0	29	29	11	140	4.5	0.032052
7	Sand	S	5	120.0	5	5	40	159	5.0	0.031369
12	Sand	S	5	120.0	5	5	69	181	5.0	0.027622
17	Rock	S	64	120.0	64	64	98	411	85.5	0.208146
								Total:	100.00	0.29919

Calculation:
Total unit weight are based on laboratory tests and/or estimated based on N_{60} values
Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)
Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i / V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively   Vs(d) = 334

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec V_{s30} = 334

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohessionless Soils

for "Sand", $V_s = \exp(4.045 + 0.096*\ln(N_{60}) + 0.236*\ln(\sigma_v')) <= 380$ m/sec

for "Silt", $V_s = \exp(3.783 + 0.178*\ln(N_{60}) + 0.231*\ln(\sigma_v')) <= 380$ m/sec

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60}=N*ER_i/60$)

Cohesive Soils ("Clay")

$V_s = \exp(3.996 + 0.230*\ln(N_{60}) + 0.164*\ln(\sigma_v')) <= 310$ m/sec

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319} <= 560$ m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}=	334.2	m/sec

Determination of V_{s30}

per Caltrans Methodology for Developing Design Response Spectrum for Seismic Design Recommendations (2012)

Project:	**Last Chance Grade**		
Project No.	20-103		
By:	PSS	Date:	10/11/2023
Check'd By:	AZ	Date:	10/20/2023

Borehole ID:	**RC-20-017**	**El. 776' approx.**
Borehole Depth:	300.00	ft
Depth to Grundwater:	150.00	ft
ER_i	60	%

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
5	Sand	S	17	120.0	17	17	29	166	7.5	0.045296
10	Sand	S	11	120.0	11	11	57	187	5.0	0.026734
15	Sand	S	28	120.0	28	28	86	225	5.0	0.022211
20	Rock	S	35	120.0	35	35	115	339	5.0	0.014757
25	Rock	S	32	120.0	32	32	144	329	5.0	0.015185
30	Rock	S	100	120.0	100	100	172	474	72.5	0.153078
								Total:	100.00	0.27726

Calculation:

Total unit weight are based on laboratory tests and/or estimated based on N_{60} values

Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)

Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i / V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively — Vs(d) = 361

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec — V_{s30} = 361

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohessionless Soils

for "Sand", $V_s = \exp(4.045 + 0.096*\ln(N_{60}) + 0.236*\ln(\sigma_v')) <= 380$m/sec

for "Silt", $V_s = \exp(3.783 + 0.178*\ln(N_{60}) + 0.231*\ln(\sigma_v')) <= 380$m/sec

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60}=N*ER_i/60$)

Cohesive Soils ("Clay")

$V_s = \exp(3.996 + 0.230*\ln(N_{60}) + 0.164*\ln(\sigma_v')) <= 310$m/sec

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319} <= 560$m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}=	**360.7**	**m/sec**

Determination of V_{s30}

per Caltrans Methodology for Developing Design Acceleration Response Spectrum for Seismic Design Recommendations (2021)

Project:	**Last Chance Grade**		
Project No.	20-103		
By:	PSS	Date:	10/11/2023
Check'd By:	AZ	Date:	10/20/2023

Borehole ID: **RC-20-016** **El. 797' approx.**

Borehole Depth: 300.00 ft

Depth to Grundwater: 136.00 ft

ER_i 60 %

1 kPa = 20.885 psf

Depth (feet)	Soil Type	Ring Type	Reduced Blow count, N (bpf)	Unit Weight* (pcf)	ASF Age	SPT-equiv. Blowcount	N_{60} (bpf)	σ_v' (kPa)	V_s (m/s)	Thickness of Layer d (ft)	d/V_s
2	Sand	S	29	120.0	Quarternary	29	29	11	114	4.5	0.039433
7	Sand	S	5	120.0	Quarternary	5	5	40	102	5.0	0.049211
12	Sand	S	5	120.0	Quarternary	5	5	69	115	5.0	0.043474
17	Rock	S	64	120.0		64	64	98	411	85.5	0.208146
									Total:	100.00	0.34026

Calculation:

Oveburden stress σ_v' is the effective overburden stress in kPa (1 kPa = 20.885 psf)

Average shear wave velocity

$V_{s(d)} = d/\{\Sigma D_i / V_{s,i}\}$

where D_i and $V_{s,i}$ are the thickness and shear wave velocity of layer i, respectively — Vs(d) = 294

If soil column depth d<30m (100 ft):

V_{s30} (m/s) = $(1.45-0.015*d)*V_{s(d)}$ in unit of m/sec — V_{s30} = 294

where $V_{s(d)}$ is in m/sec

Correlations between V_s and SPT-N value

Cohesive Soils ("Clay")

$V_s = 26*(ASF)*(N_{60})$^0.17*(σ_v')^0.32)

Where, ASF =1.0 for Quaternary or 0.88 for Holocene and 1.12 for Pleistocene.

where N_{60} is the SPT-N value corrected only for the hammer energy (i.e., $N_{60}=N*ER_i/60$)

Cohessionless Soils ("Sand" and "Silt")

For "Silt", The SPT N60 correlation recommended above for cohesive soil layers is also recommended for silt layers.

For "Sand", $V_s = 30*(ASF)*(N_{60})$^0.23*(σ_v')^0.23)

Where, ASF =1.0 for Quaternary or 0.9 for Holocene and 1.17 for Pleistocene.

For "Gravel" $V_s = 53*(N_{60})$^0.19*(σ_v')^0.18) for Holocene

$V_s = 115*(N_{60})$^0.17*(σ_v')^0.12) for Pleistocene

Young Sedimentary Rock (Tertiary deposits) ("Rock")

$V_s = 109*(N_{60})^{0.319}$ <= 560m/sec

Competent Rock

V_s = f(hardness, fracture) <= 760m/sec

Resulting shear wave velocity

V_{s30}= 293.9 m/sec

APPENDIX B2 ARS Online Outputs

ARS Online V3.1.0

Retaining Wall 6

Using the tool: Specify latitude and longitude in decimal degrees in the input boxes below. Specify the time-averaged shear-wave velocity in the upper 30m (Vs30) in the input box. After submitting the data, the USGS 2014 hazard data for a 975-year return period will be reported along with adjustment factors required by Caltrans Seismic Design Criteria (SDC) V2.0.

Latitude: 41.6328 **Longitude:** -124.1146 **Vs30 (m/s):** 310

Submit

Caltrans Design Spectrum (5% damping)

Period(s)	Sa_{2014}(g)	$Basin_{2014}$	Near Fault Amp	Design Sa_{2014}(g)
PGA	0.87	1	1	0.87
0.10	1.43	1	1	1.43
0.20	1.77	1	1	1.77
0.30	1.88	1	1	1.88
0.50	1.68	1	1	1.68
0.75	1.36	1	1.05	1.42
1.0	1.07	1	1.1	1.18
2.0	0.55	1	1.1	0.61
3.0	0.33	1	1.1	0.37
4.0	0.23	1	1.1	0.25
5.0	0.16	1	1.1	0.17

Copy table

Deaggregation (based on 2014 hazard)

Mean moment magnitude (for PGA)	8.66
Mean site-source distance, km (for Sa at 1s)	20

Option: recalculate Near Fault amplification with user specified distance

Site-source distance, km: 20 Update

ARS Online V3.1.0

Retaining Wall 7A-1

Using the tool: Specify latitude and longitude in decimal degrees in the input boxes below. Specify the time-averaged shear-wave velocity in the upper 30m (Vs30) in the input box. After submitting the data, the USGS 2014 hazard data for a 975-year return period will be reported along with adjustment factors required by Caltrans Seismic Design Criteria (SDC) V2.0.

Latitude: 41.6263 **Longitude:** -124.1124 **Vs30 (m/s):** 280

Submit

Caltrans Design Spectrum (5% damping)

Period(s)	Sa_{2014}(g)	$Basin_{2014}$	Near Fault Amp	Design Sa_{2014}(g)
PGA	0.88	1	1	0.88
0.10	1.4	1	1	1.4
0.20	1.73	1	1	1.73
0.30	1.89	1	1	1.89
0.50	1.76	1	1	1.76
0.75	1.44	1	1.05	1.51
1.0	1.13	1	1.1	1.25
2.0	0.6	1	1.1	0.66
3.0	0.36	1	1.1	0.4
4.0	0.24	1	1.1	0.27
5.0	0.17	1	1.1	0.18

Copy table

Deaggregation (based on 2014 hazard)

Mean moment magnitude (for PGA)	8.65
Mean site-source distance, km (for Sa at 1s)	20

Option: recalculate Near Fault amplification with user specified distance

Site-source distance, km: 20 Update

CA.GOV

ARS Online V3.1.0

Retaining Wall 7A-2

Using the tool: Specify latitude and longitude in decimal degrees in the input boxes below. Specify the time-averaged shear-wave velocity in the upper 30m (Vs30) in the input box. After submitting the data, the USGS 2014 hazard data for a 975-year return period will be reported along with adjustment factors required by Caltrans Seismic Design Criteria (SDC) V2.0.

Latitude: 41.6309 **Longitude:** -124.1132 **Vs30 (m/s):** 310

Submit

Caltrans Design Spectrum (5% damping)

Period(s)	Sa_{2014}(g)	$Basin_{2014}$	Near Fault Amp	Design Sa_{2014}(g)
PGA	0.87	1	1	0.87
0.10	1.43	1	1	1.43
0.20	1.77	1	1	1.77
0.30	1.88	1	1	1.88
0.50	1.68	1	1	1.68
0.75	1.36	1	1.05	1.42
1.0	1.07	1	1.1	1.18
2.0	0.55	1	1.1	0.61
3.0	0.33	1	1.1	0.37
4.0	0.23	1	1.1	0.25
5.0	0.16	1	1.1	0.17

Copy table

Deaggregation (based on 2014 hazard)

Mean moment magnitude (for PGA)	8.66
Mean site-source distance, km (for Sa at 1s)	20

Option: recalculate Near Fault amplification with user specified distance

Site-source distance, km: 20 Update

CA.GOV

ARS Online V3.1.0

Retaining Wall 7A-3

Using the tool: Specify latitude and longitude in decimal degrees in the input boxes below. Specify the time-averaged shear-wave velocity in the upper 30m (Vs30) in the input box. After submitting the data, the USGS 2014 hazard data for a 975-year return period will be reported along with adjustment factors required by Caltrans Seismic Design Criteria (SDC) V2.0.

Latitude: 41.6393 **Longitude:** -124.1152 **Vs30 (m/s):** 310

Submit

Caltrans Design Spectrum (5% damping)

Period(s)	Sa_{2014}(g)	$Basin_{2014}$	Near Fault Amp	Design Sa_{2014}(g)
PGA	0.87	1	1	0.87
0.10	1.43	1	1	1.43
0.20	1.76	1	1	1.76
0.30	1.87	1	1	1.87
0.50	1.67	1	1	1.67
0.75	1.35	1	1.05	1.42
1.0	1.07	1	1.1	1.18
2.0	0.55	1	1.1	0.61
3.0	0.33	1	1.1	0.37
4.0	0.22	1	1.1	0.25
5.0	0.15	1	1.1	0.17

Copy table

Deaggregation (based on 2014 hazard)

Mean moment magnitude (for PGA)	8.66
Mean site-source distance, km (for Sa at 1s)	20

Option: recalculate Near Fault amplification with user specified distance

Site-source distance, km: 20 Update

.GOV

ARS Online V3.1.0

Retaining Wall 7B/7C

Using the tool: Specify latitude and longitude in decimal degrees in the input boxes below. Specify the time-averaged shear-wave velocity in the upper 30m (Vs30) in the input box. After submitting the data, the USGS 2014 hazard data for a 975-year return period will be reported along with adjustment factors required by Caltrans Seismic Design Criteria (SDC) V2.0.

Latitude: 41.6378 **Longitude:** -124.1150 **Vs30 (m/s):** 320

Submit

Caltrans Design Spectrum (5% damping)

Period(s)	Sa_{2014}(g)	$Basin_{2014}$	Near Fault Amp	Design Sa_{2014}(g)
PGA	0.86	1	1	0.86
0.10	1.43	1	1	1.43
0.20	1.77	1	1	1.77
0.30	1.86	1	1	1.86
0.50	1.65	1	1	1.65
0.75	1.32	1	1.05	1.39
1.0	1.05	1	1.1	1.15
2.0	0.54	1	1.1	0.59
3.0	0.32	1	1.1	0.36
4.0	0.22	1	1.1	0.24
5.0	0.15	1	1.1	0.17

Copy table

Deaggregation (based on 2014 hazard)

Mean moment magnitude (for PGA)	8.66
Mean site-source distance, km (for Sa at 1s)	20

Option: recalculate Near Fault amplification with user specified distance

Site-source distance, km: 20 Update

ARS Online V3.0.2

Retaining Wall 1

Using the tool: Specify latitude and longitude in decimal degrees in the input boxes below. Alternatively, **Google Maps** can be used to find the site location. Specify the time-averaged shear-wave velocity in the upper 30m (Vs30) in the input box. After submitting the data, the USGS 2014 hazard data for a 975-year return period will be reported along with adjustment factors required by Caltrans Seismic Design Criteria (SDC) V2.0.

Latitude: 41.6247 **Longitude:** -124.1115 **Vs30 (m/s):** 280 Submit

Caltrans Design Spectrum (5% damping)

Period(s)	Sa_{2008}(g)	Sa_{2014}(g)	$Basin_{2008}$	$Basin_{2014}$	Near Fault Amp	Design Sa_{2008}(g)	Design Sa_{2014}(g)
PGA	0.51	0.88	1	1	1	0.51	0.88
0.10	0.86	1.4	1	1	1	0.86	1.4
0.20	1.17	1.73	1	1	1	1.17	1.73
0.30	1.26	1.89	1	1	1	1.26	1.89
0.50	1.09	1.76	1	1	1	1.09	1.76
0.75	0.83	1.44	1	1	1.05	0.87	1.51
1.0	0.63	1.13	1	1	1.1	0.7	1.25
2.0	0.31	0.6	1	1	1.1	0.35	0.66
3.0	0.19	0.36	1	1	1.1	0.2	0.4
4.0	0.13	0.24	1	1	1.1	0.15	0.27
5.0	0.09	0.17	1	1	1.1	0.1	0.18

Copy table

Deaggregation (based on 2014 hazard)

mean magnitude (for PGA)	8.65
mean site-source distance (km, for Sa at 1s)	20

Option: recalculate Near Fault amplification with user specified distance

Site-source distance (km): 20 Update

CA

ARS Online V3.0.2

Tunnel South Portal-RW 2R/2L

Using the tool: Specify latitude and longitude in decimal de
Alternatively, **Google Maps** can be used to find the site location. Specify the time-averaged shear-wave velocity in the upper 30m (Vs30) in the input box. After submitting the data, the USGS 2014 hazard data for a 975-year return period will be reported along with adjustment factors required by Caltrans Seismic Design Criteria (SDC) V2.0.

Latitude: 41.6262 **Longitude:** -124.1109 **Vs30 (m/s):** 280 Submit

Caltrans Design Spectrum (5% damping)

Period(s)	Sa_{2008}(g)	Sa_{2014}(g)	$Basin_{2008}$	$Basin_{2014}$	Near Fault Amp	Design Sa_{2008}(g)	Design Sa_{2014}(g)
PGA	0.51	0.87	1	1	1	0.51	0.87
0.10	0.86	1.4	1	1	1	0.86	1.4
0.20	1.17	1.73	1	1	1	1.17	1.73
0.30	1.26	1.89	1	1	1	1.26	1.89
0.50	1.09	1.76	1	1	1	1.09	1.76
0.75	0.83	1.43	1	1	1.05	0.87	1.51
1.0	0.63	1.13	1	1	1.1	0.7	1.24
2.0	0.31	0.6	1	1	1.1	0.35	0.66
3.0	0.19	0.36	1	1	1.1	0.2	0.4
4.0	0.13	0.24	1	1	1.1	0.15	0.27
5.0	0.09	0.17	1	1	1.1	0.1	0.18

Copy table

Deaggregation (based on 2014 hazard)

mean magnitude (for PGA)	8.65
mean site-source distance (km, for Sa at 1s)	20

Option: recalculate Near Fault amplification with user specified distance

Site-source distance (km): 20 Update

ARS Online V3.0.2

Tunnel - Middle

Using the tool: Specify latitude and longitude in decimal de
Alternatively, **Google Maps** can be used to find the site location. Specify the time-averaged shear-wave velocity in the upper 30m (Vs30) in the input box. After submitting the data, the USGS 2014 hazard data for a 975-year return period will be reported along with adjustment factors required by Caltrans Seismic Design Criteria (SDC) V2.0.

Latitude: 41.6344 **Longitude:** -124.1105 **Vs30 (m/s):** 1149 Submit

Caltrans Design Spectrum (5% damping)

Period(s)	Sa_{2008}(g)	Sa_{2014}(g)	$Basin_{2008}$	$Basin_{2014}$	Near Fault Amp	Design Sa_{2008}(g)	Design Sa_{2014}(g)
PGA	0.35	0.65	1	1	1	0.35	0.65
0.10	0.69	1.39	1	1	1	0.69	1.39
0.20	0.79	1.32	1	1	1	0.79	1.32
0.30	0.7	1.09	1	1	1	0.7	1.09
0.50	0.46	0.77	1	1	1	0.46	0.77
0.75	0.34	0.57	1	1	1.05	0.35	0.6
1.0	0.3	0.46	1	1	1.09	0.33	0.5
2.0	0.15	0.24	1	1	1.09	0.17	0.26
3.0	0.07	0.15	1	1	1.09	0.08	0.16
4.0	0.05	0.1	1	1	1.09	0.06	0.11
5.0	0.03	0.07	1	1	1.09	0.04	0.08

Copy table

Deaggregation (based on 2014 hazard)

mean magnitude (for PGA)	8.58
mean site-source distance (km, for Sa at 1s)	20.3

Option: recalculate Near Fault amplification with user specified distance

Site-source distance (km): 20.3 Update

ARS Online V3.0.2

Tunnel North Portal-RW 3R/3L

Using the tool: Specify latitude and longitude in decimal de
Alternatively, **Google Maps** can be used to find the site location. Specify the time-averaged shear-wave velocity in the upper 30m (Vs30) in the input box. After submitting the data, the USGS 2014 hazard data for a 975-year return period will be reported along with adjustment factors required by Caltrans Seismic Design Criteria (SDC) V2.0.

Latitude: 41.6425 **Longitude:** -124.1146 **Vs30 (m/s):** 340 Submit

Caltrans Design Spectrum (5% damping)

Period(s)	Sa_{2008}(g)	Sa_{2014}(g)	$Basin_{2008}$	$Basin_{2014}$	Near Fault Amp	Design Sa_{2008}(g)	Design Sa_{2014}(g)
PGA	0.5	0.85	1	1	1	0.5	0.85
0.10	0.88	1.43	1	1	1	0.88	1.43
0.20	1.17	1.76	1	1	1	1.17	1.76
0.30	1.25	1.82	1	1	1	1.25	1.82
0.50	1.05	1.57	1	1	1	1.05	1.57
0.75	0.79	1.25	1	1	1.05	0.83	1.31
1.0	0.59	1	1	1	1.1	0.65	1.1
2.0	0.28	0.51	1	1	1.1	0.31	0.56
3.0	0.16	0.31	1	1	1.1	0.18	0.34
4.0	0.12	0.21	1	1	1.1	0.13	0.23
5.0	0.08	0.14	1	1	1.1	0.08	0.16

Copy table

Deaggregation (based on 2014 hazard)

mean magnitude (for PGA)	8.67
mean site-source distance (km, for Sa at 1s)	20.1

Option: recalculate Near Fault amplification with user specified distance

Site-source distance (km): 20.1 Update

APPENDIX C Alternative F-Short Feasibility Study Analyses

Last Chance Grade

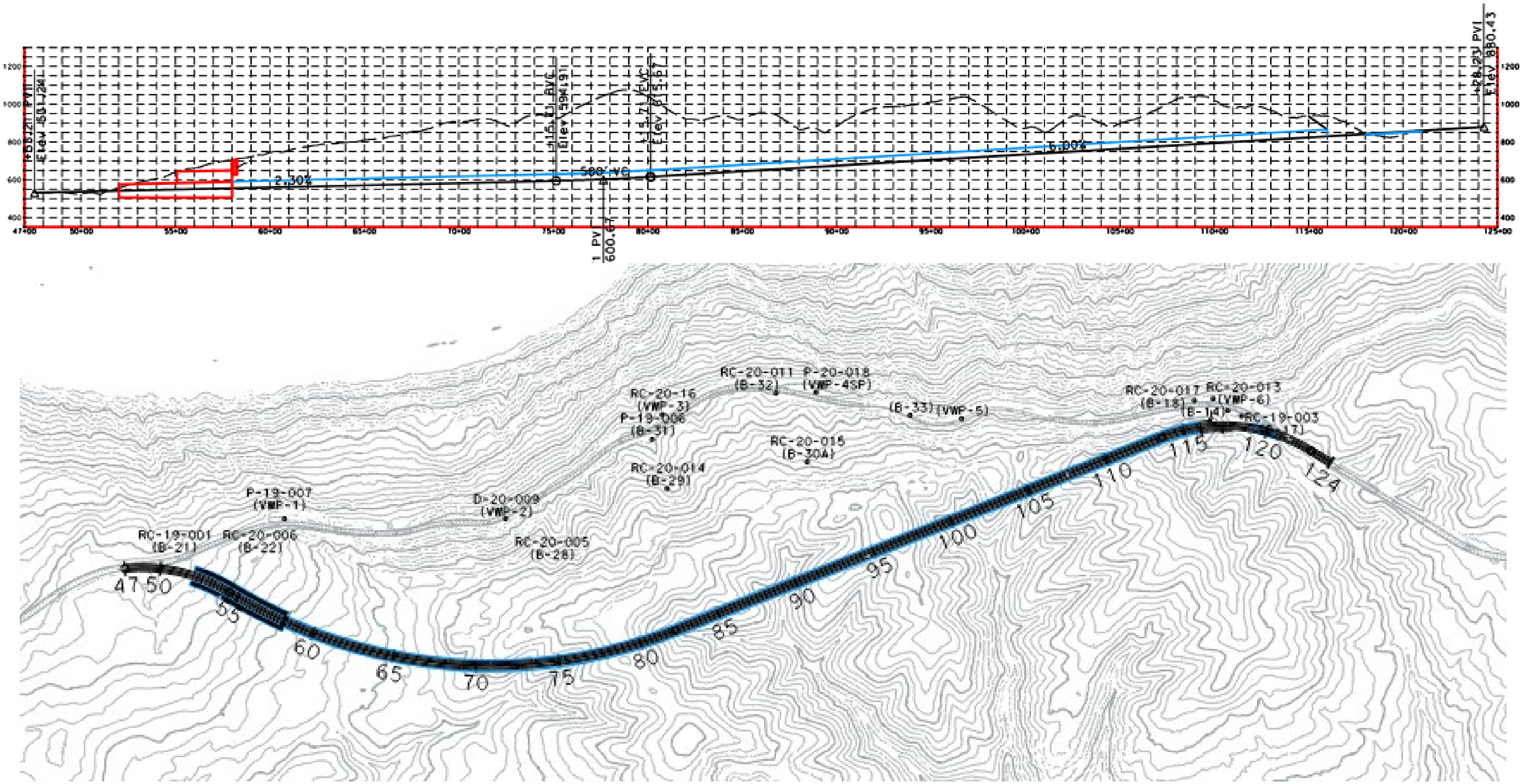
Alternative F - Short Design Analyses

HNTB

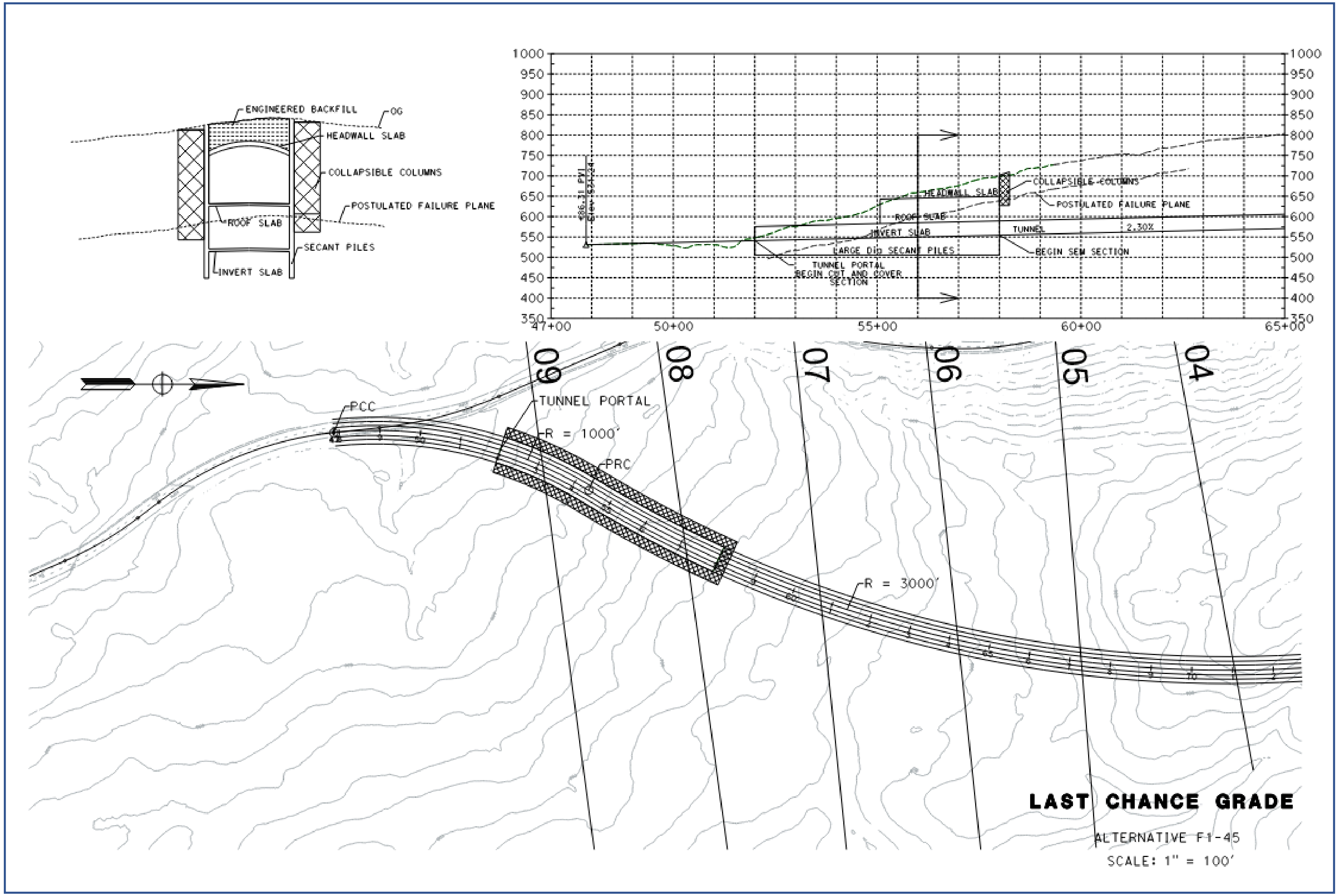
10-27-2023

1. Alignment - Alternative F1 Short
2. South Portal
    2.1 Driving force on retaining walls
    2.2 Crushable Cellular Concrete Columns (EDAS)
    2.3 Design of Retaining Walls
3. Tunnel Design
    Seismic Ovaling Analysis and P-M Diagrams
4. Findings

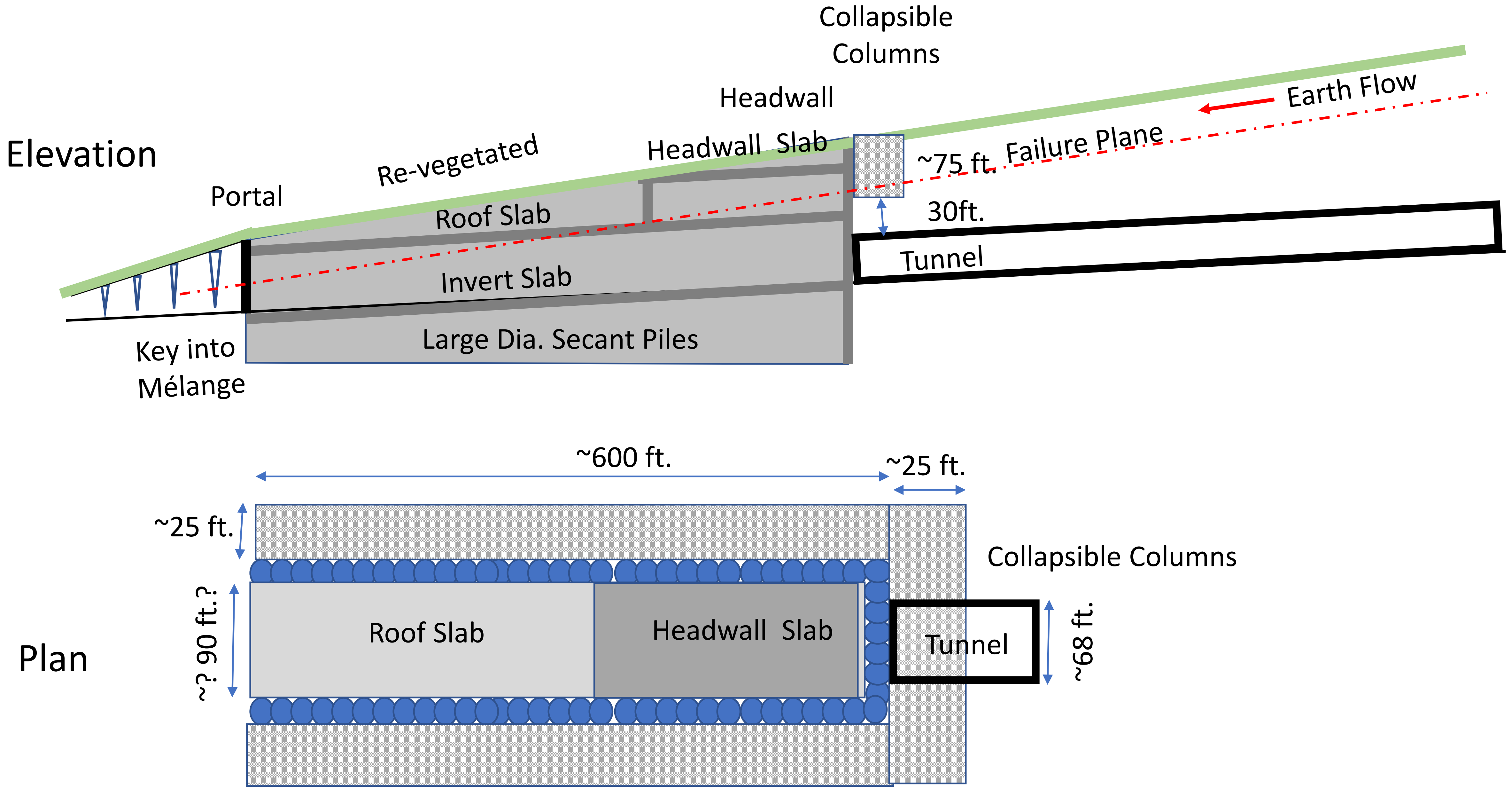
Alignment - Alternative F Short



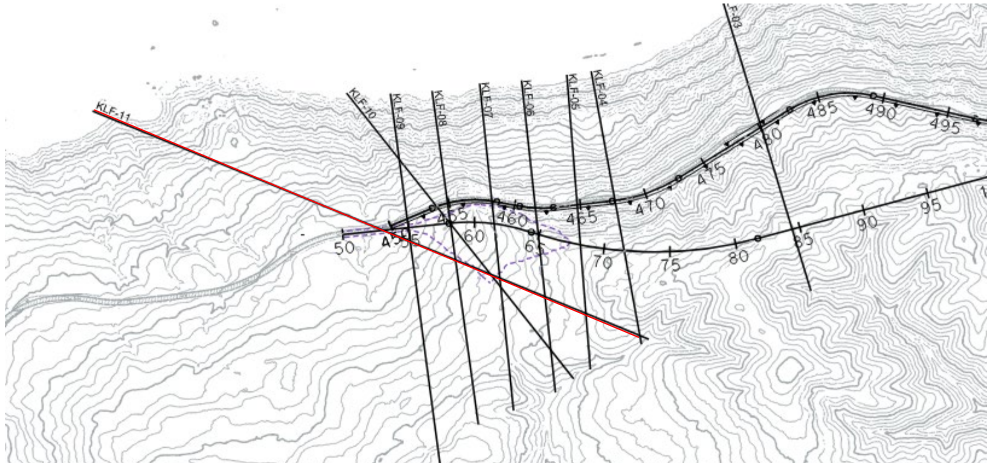
South Approach Structure
Plan and Sections

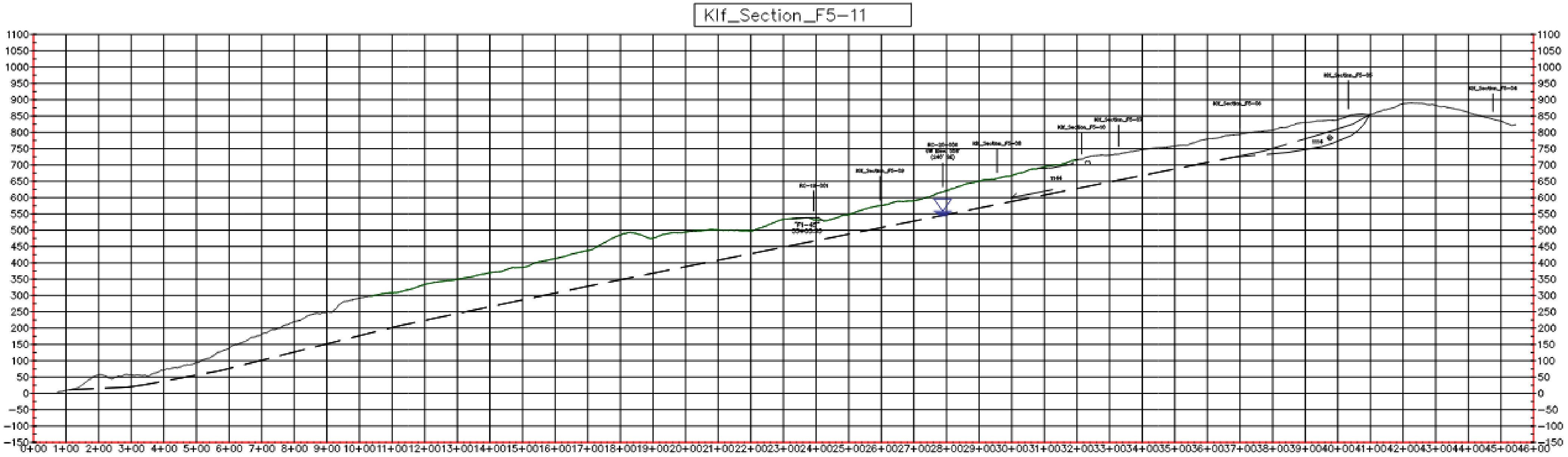


EDAS - Load Moderating System at South Portal



Section 11 Used to Assess EDAS





Geologic Units and Strength Parameters

Table X. Geologic Units and Strength Parameters

Symbol	Geologic Unit	Description	Lithology for Analysis	Total Unit Weight	Internal Friction Angle (ϕ)	Cohesion (c)
				pcf	degrees	psf
Qal	Alluvium	Sand and sandy gravel with some fine-grained soil *(confirm)*	*(not used for analysis?)*	-	-	-
Qc	Colluvium	Loose, heterogeneous mass of soil material and/or rock fragments transported and deposited downslope by sheet flow or slow, continuous creep	mixed (gravel to clay)	125 $\boxed{120}$	26 $\boxed{25}$	50 $\boxed{50}$
Qlsd-m	Earth Flow Landslide Deposits, Derived from Mélange	Landslide deposits consisting of a mixture of fine-grained soils, deeply weathered rock, and scattered sandstone clasts which have been transported as a sliding mass with many internal slip surfaces	argillite with sandstone clasts	140 $\boxed{130}$	28 $\boxed{26}$	250* $\boxed{250}$
Qlsd-bf	Rock/Debris Landslide Deposits, Derived from Broken Formation	Landslide deposits consisting of blocks of sandstone and argillite rock and/or debris which have been transported by sliding or falling	sandstone/argillite	140 $\boxed{140}$	36 $\boxed{45}$	1500* $\boxed{1000 - 3000}$
Rs-1	Earth Flow Basal Failure Zone(s)	Discrete clay-rich sliding zone at the base of Earth Flow Landslide Deposits	sheared argillite	145	18*	0
Rs-2	Rock/Debris Landslide Failure Zone(s)	Narrow, sheared zones of weakness along which sliding occurs within and at the base of Rock/Debris Landslide Deposits	sheared sandstone/argillite	145	32 to 34*	0
KJFm	Franciscan Complex, Mélange	Dark gray, pervasively sheared, soil-like argillite with scattered blocks of intact sandstone	argillite	145	28	500
KJFbf	Franciscan Complex, Broken Formation	Blocks of gray, hard, thick-bedded sandstone with interbedded argillite separated by weak, sheared zones	interlayered sandstone/argillite	155	40	1500*

Note: *Strength parameters estimated from back-calculation on existing slope configurations and groundwater conditions.

Engineering Properties

At South Portal and Tunnel Sections

Soil Name	Formation	Geologic Unit	Unit Weight, pcf	Friction Angle (deg)	Cohesion (psf)
Soil	Qlsd-m	Earth Flow Landslide,	135	28	250
Slipping Zone	Rs-1	Earth Flow Basal Failure Zone(s)	135	18	0
Soft Rock	KJFm	Franciscan Complex, Melange	145	28	500

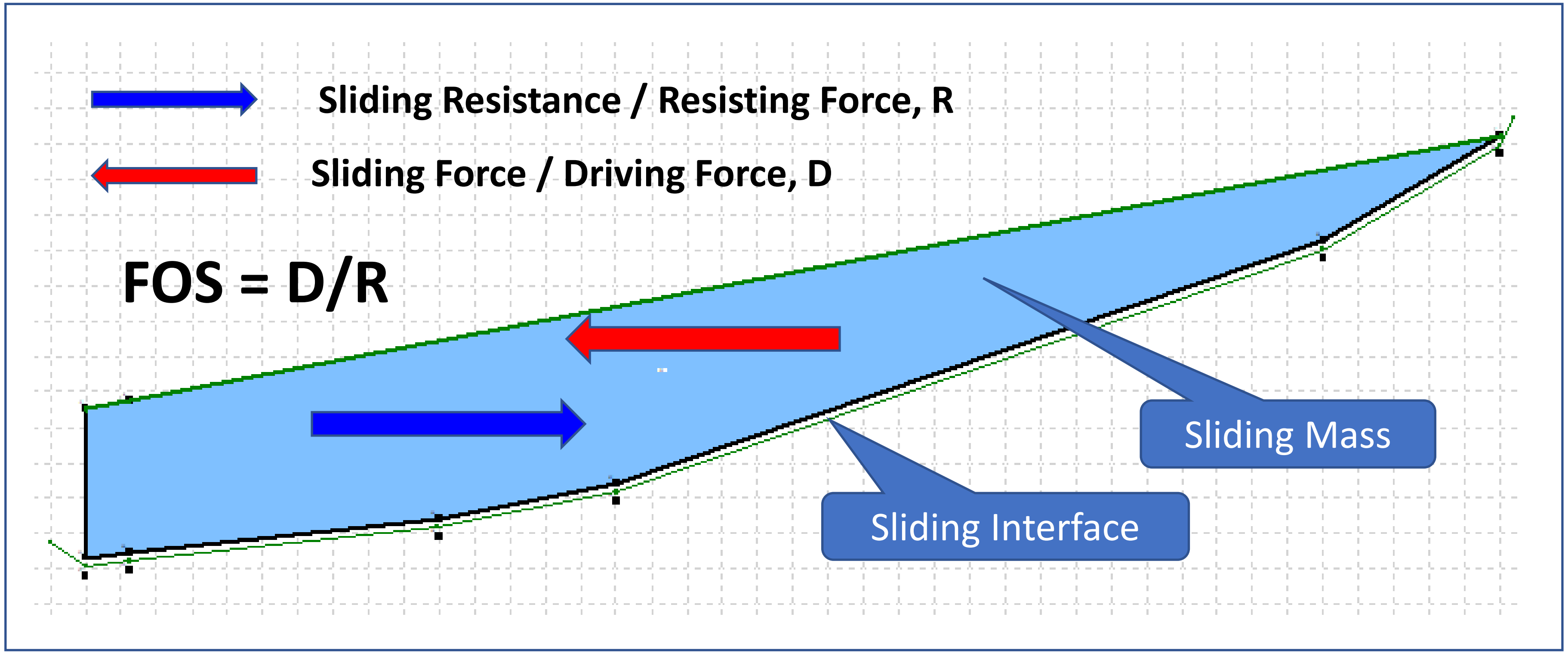
At North Portal Section

Soil Name	Formation	Geologic Unit	Unit Weight, pcf	Friction Angle (deg)	Cohesion (psf)
Soil	Qlsd-bf	Rock / Debris Landslide	140	36	1500
Soft Rock	KJbf	Franciscan Complex, Broken Formation	155	40	1500

2.1 South Portal

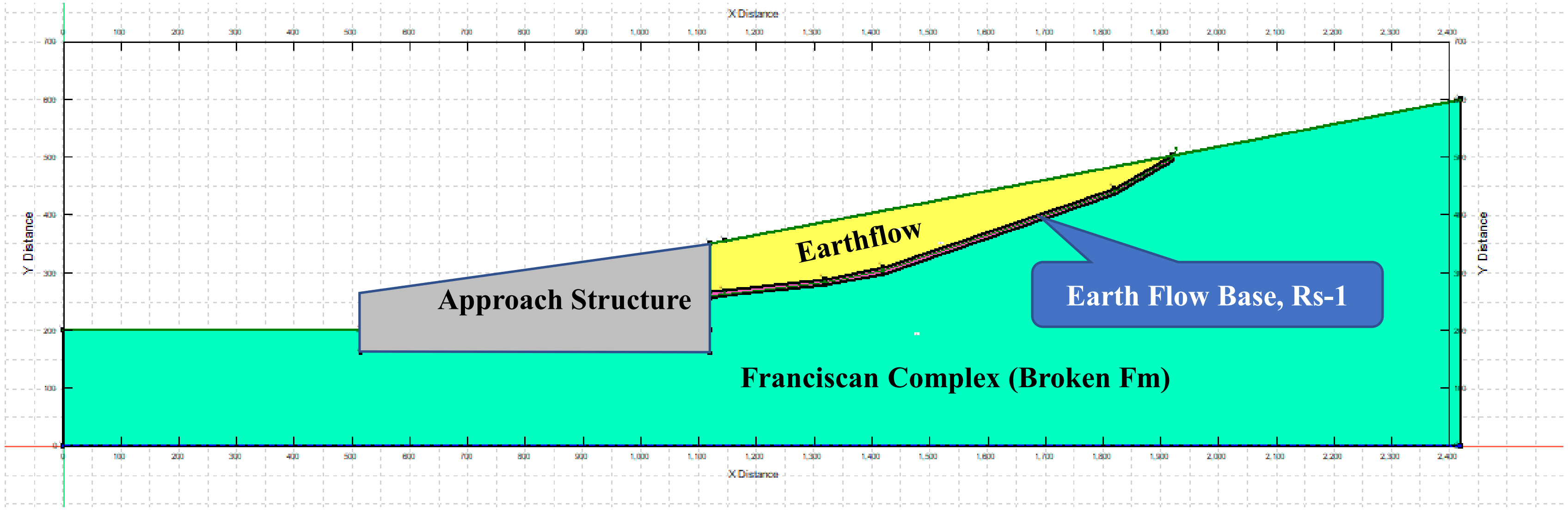
Development of lateral earth pressures acting on south approach structure

Derivation of Driving Forces from Earth Flow



SLOPE/W Model

Geologic Unit	Formation	Unit Weight, (pcf)	Friction Angle (deg)	Cohesion (psf)
Earth Flow Landslide,	Qlsd-m	135	28	250
Earth Flow Basal Failure Zone(s)	Rs-1	135	18	0
Franciscan Complex, Melange	KJFm	145	28	500



Summary of SLOPE/W Analysis

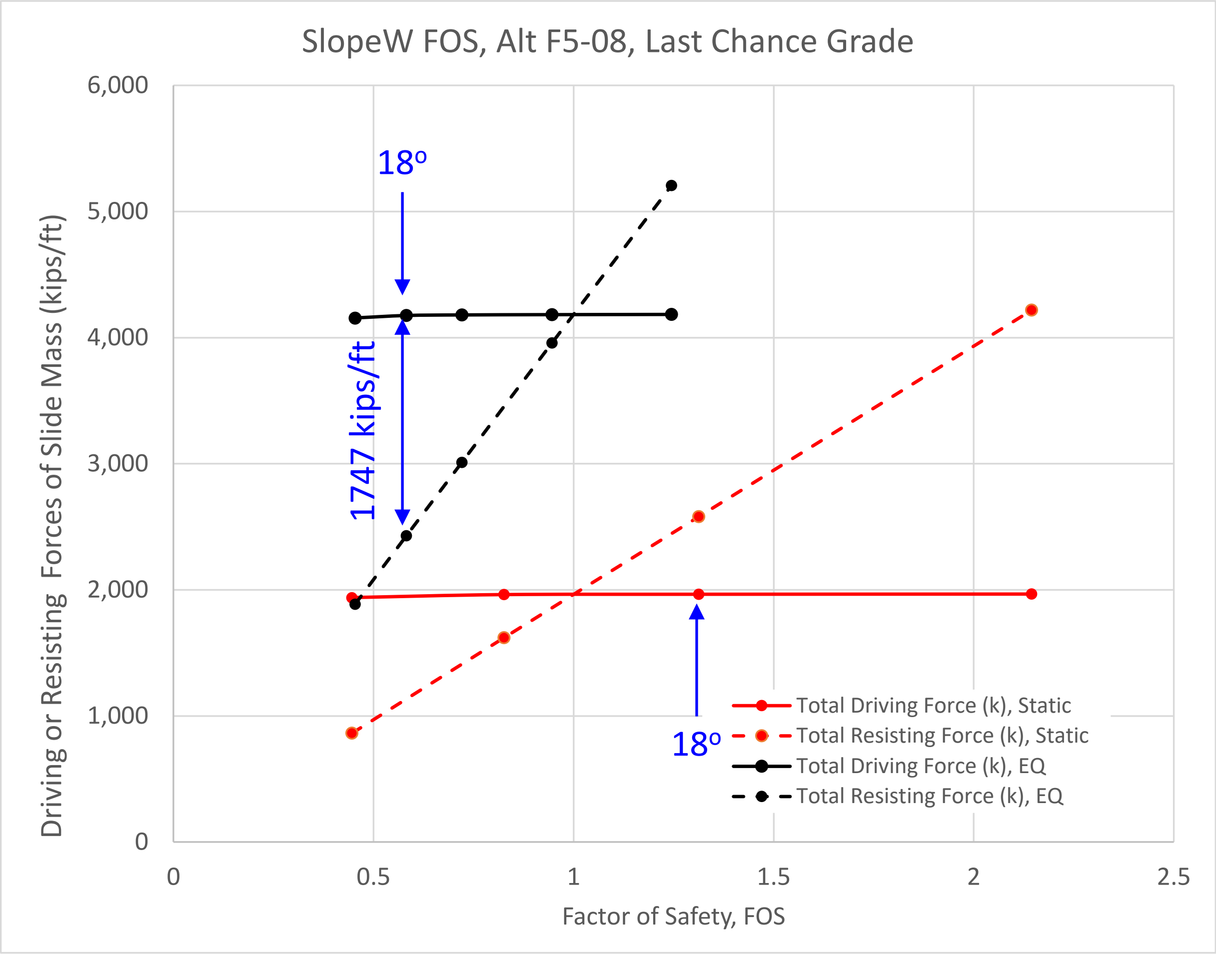
Static Condition

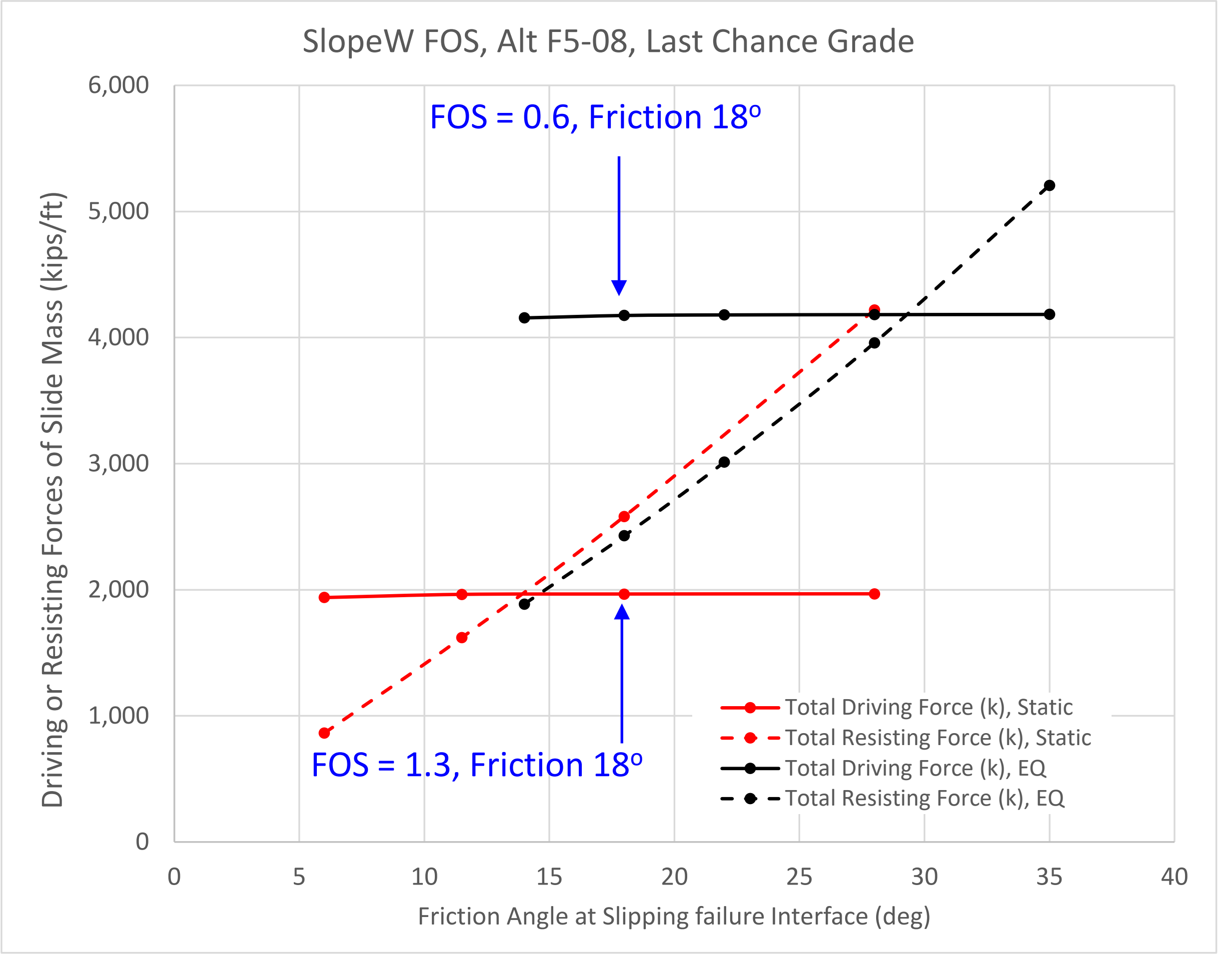
FOS	Friction Angle of Failure Zone (deg)	Total Driving Force (kips/ft), Static	Total Resisting Force (kips/ft), Static	Net Force (kips/ft), Static
0.446	6	1,939	864	75
0.826	11.5	1,963	1,621	342
1.313	18	1,966	2,582	-616
2.145	28	1,967	4,219	-2,252

Earthquake Condition with k_h = 0.29g (i.e., 1/3 PGA = 0.87g/3)

FOS	Friction Angle of Failure Zone (deg)	Total Driving Force (kips/ft), Earthquake	Total Resisting Force (kips/ft), Earthquake	Net Force (kips/ft), Earthquake
0.454	14	4,156	1,887	2,269
0.582	18	4,176	2,429	1,747
0.721	22	4,180	3,012	1,168
0.946	28	4,182	3,958	224
1.245	35	4,184	5,207	-1,023

Lateral Forces from Earth Flow





- Estimated Net Earth Flow Load is approximately 1,500 kips/ft during earthquake conditions.

2.2 South Portal

Imposed earth flow loads with benefit of collapsible columns (EDAS using cellular concrete)

Typical Cellular Concrete Behavior

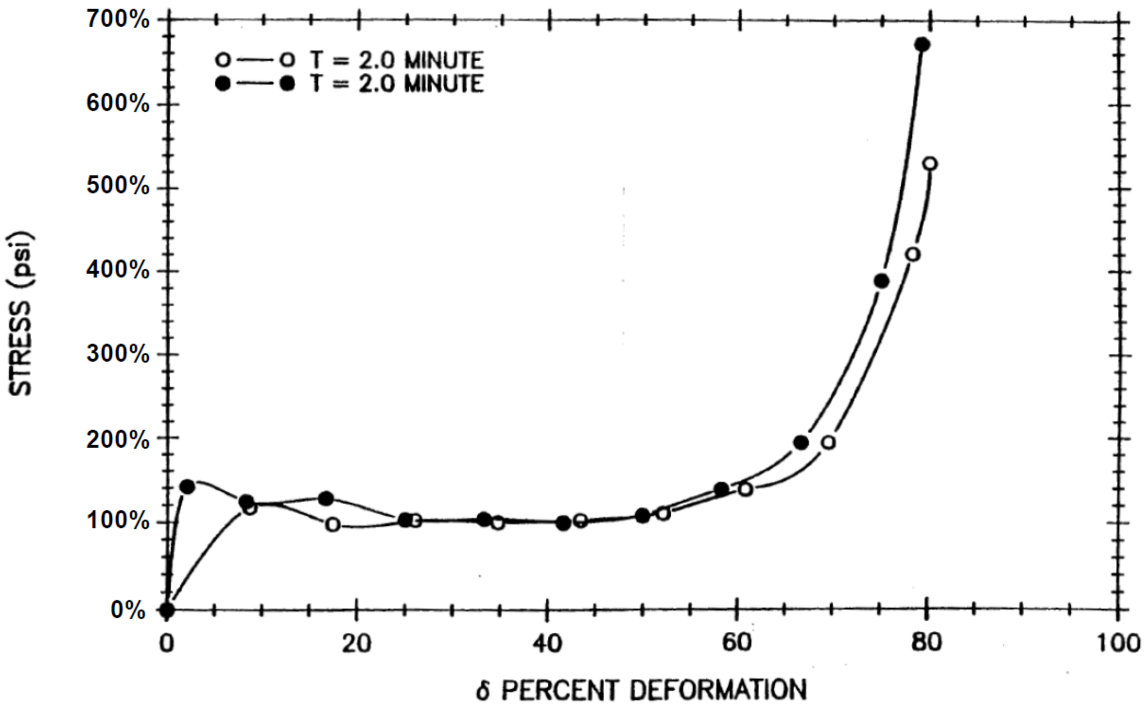


***Figure 2-1. Uniaxial Compression Load Curve for Foam Concrete*[4]**

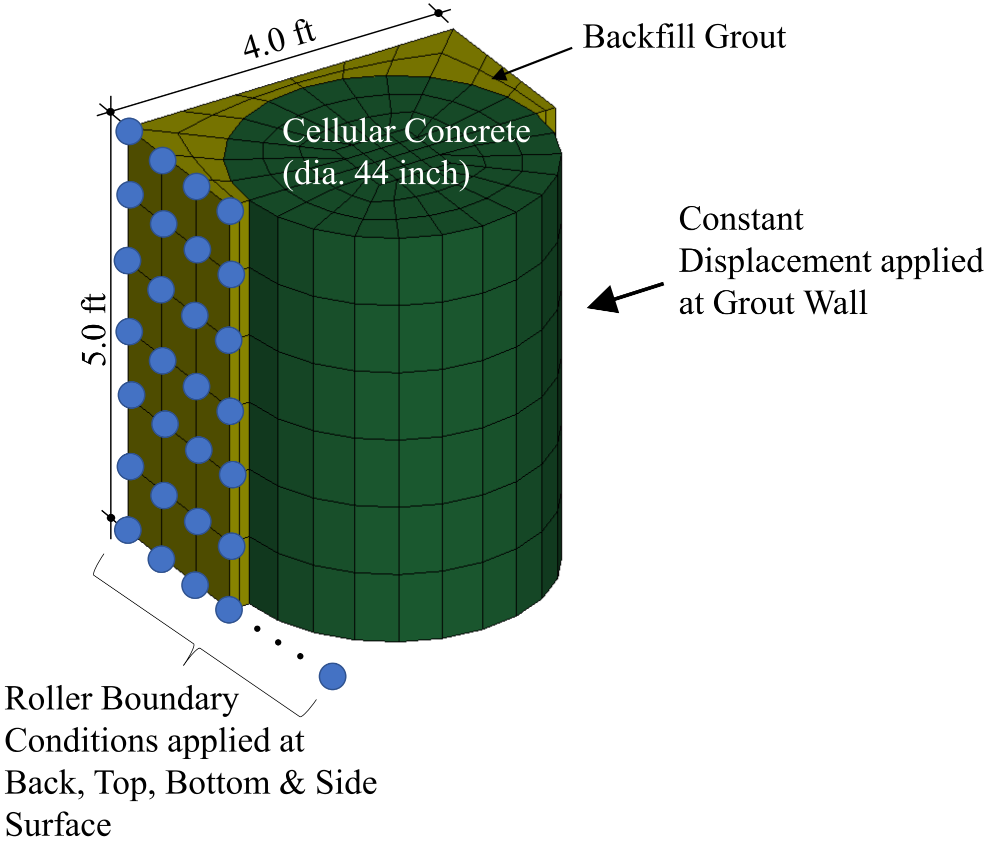
Table 5-3. Parameters for *MAT_163 or *MAT_MODIFIED_CRUSHABLE_FOAM

Parameter	Symbol	Description
MID		Material ID number
RO	ρ	Density
E	E	Young's modulus
PR	ν	Poisson's ratio
TID		Table ID defining yield stress versus volumetric strain
TSC		Tensile stress cutoff
DAMP		Rate sensitivity via damping coefficient
NCYCLE		Number of cycles to determine average volumetric strain rate
SRCLMT		Strain rate change limit

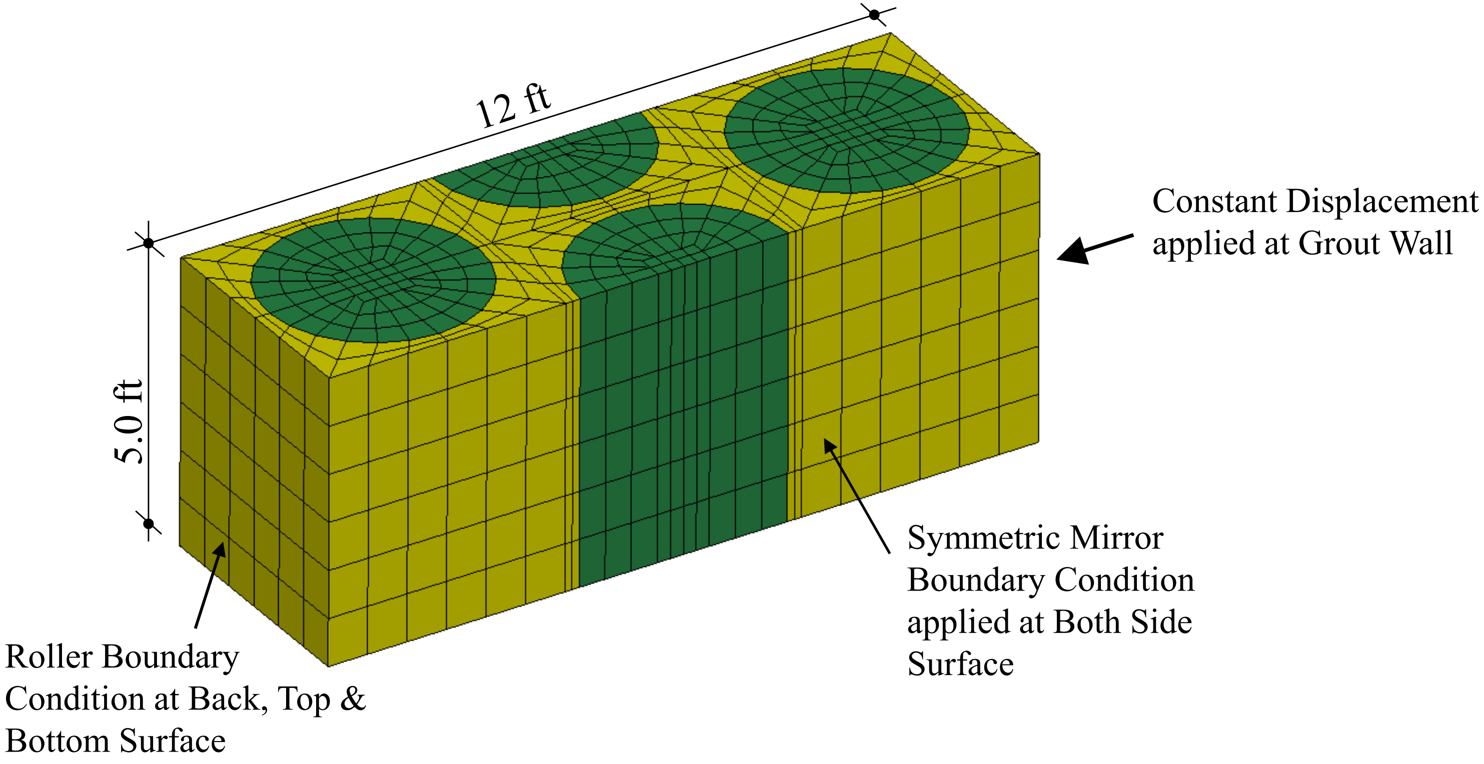
[4] Figure appeared in (Cook R. F., 1988) as Figure 45, "Concrete Foam Crushing Strength Characteristics," Density = 25 pcf. Stress values have been normalized against plateau value.

Cellular Concrete Model using LS-DYNA

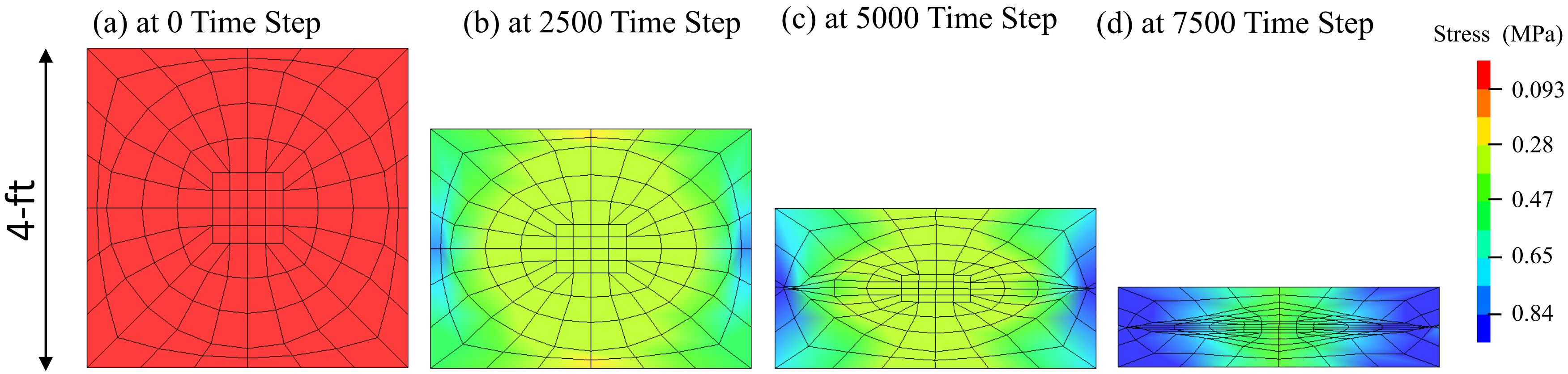
4-ft Thick Cellular Concrete Column



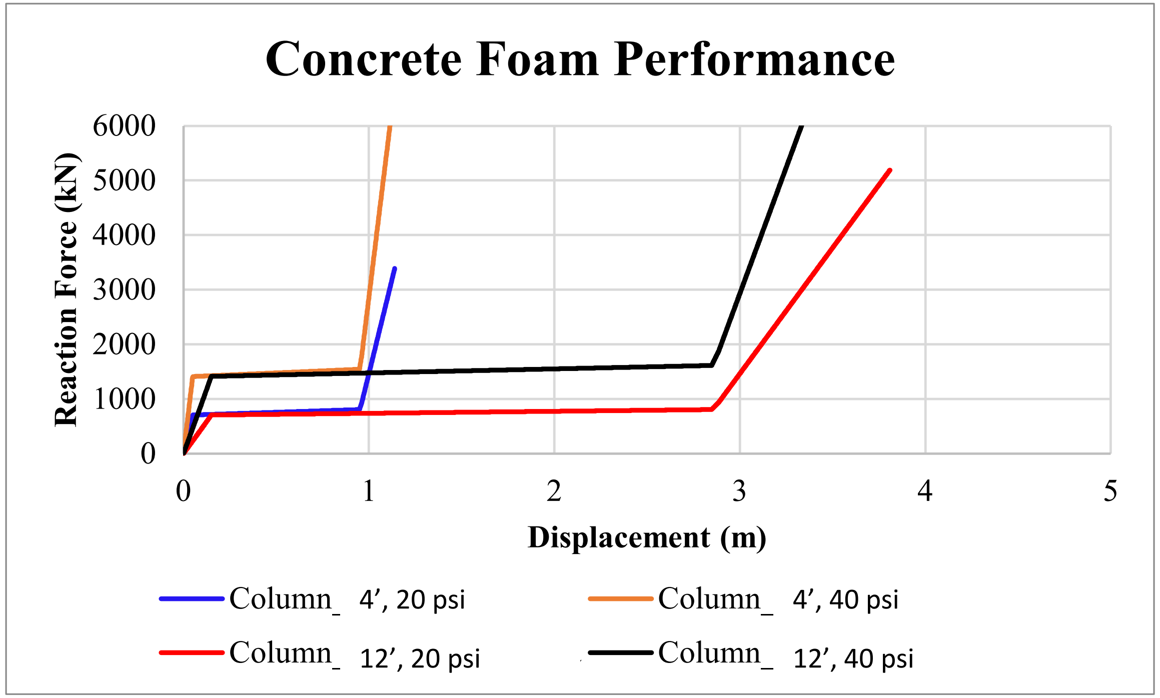
12-ft Thick Cellular Concrete Column



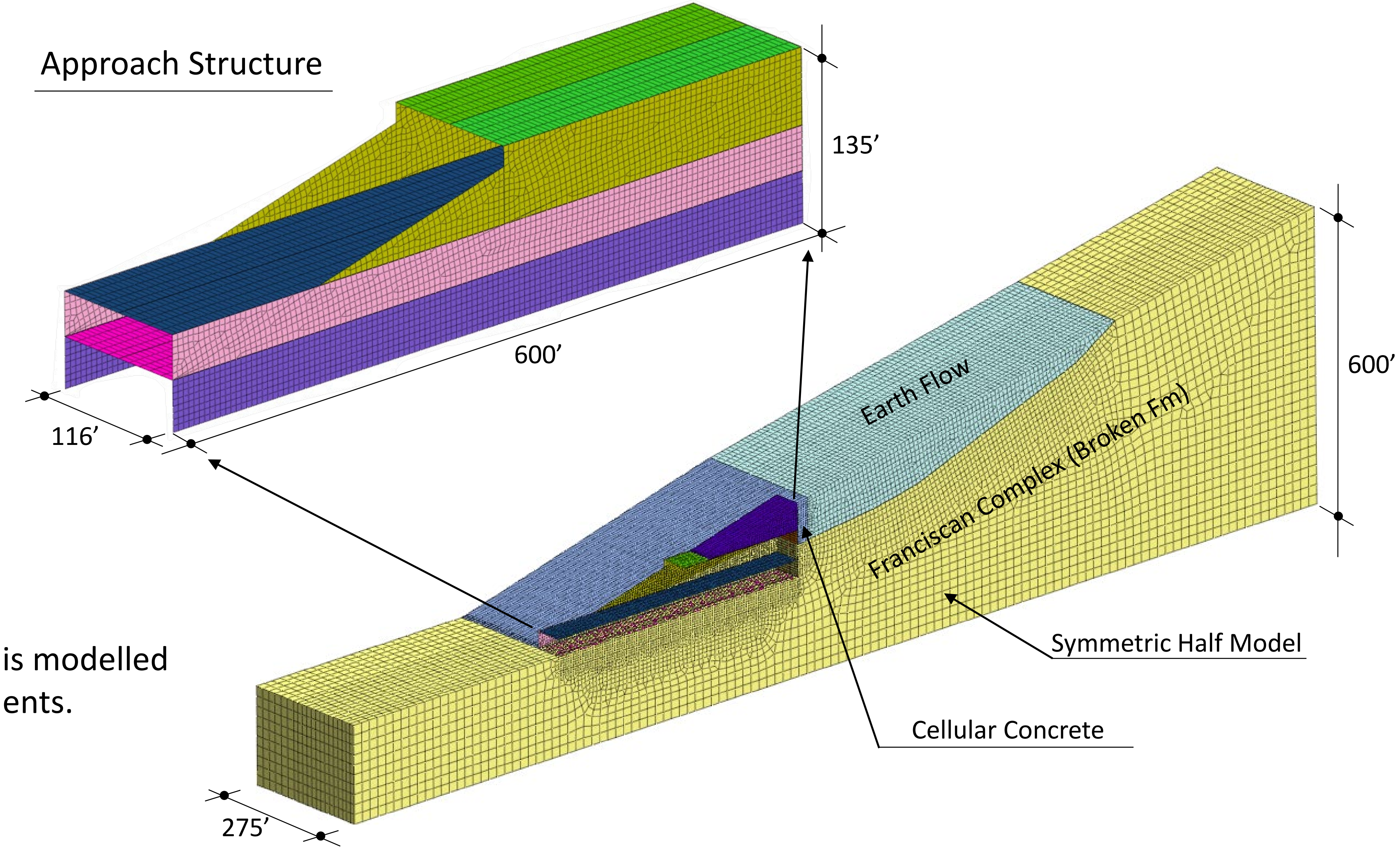
Results of LS-DYNA Analysis



fc' (psi)	E (ksi)	Strain at Plateau	Plateau Strength (psi) at 0.75 strain	tensile strength (10% of fc')
20	11	0.00175	23	2
40	16	0.00247	46	4
60	56	0.00107	69	6
100	101	0.00099	115	10
500	297	0.00168	575	50
1100	810	0.00136	1265	110

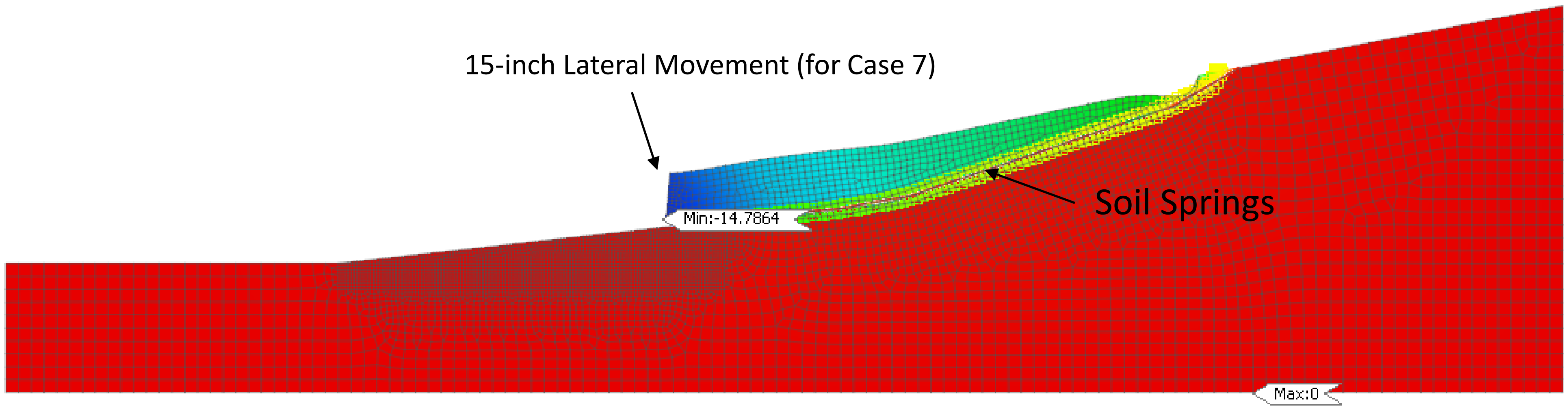


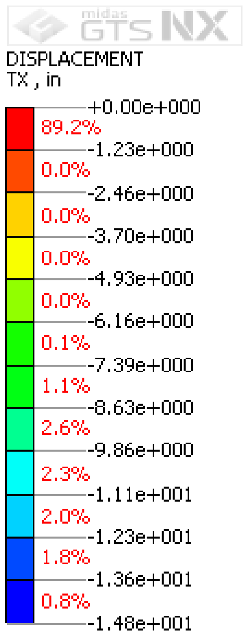
FE Model Configuration



Note: The cellular concrete is modelled with non-linear spring elements.

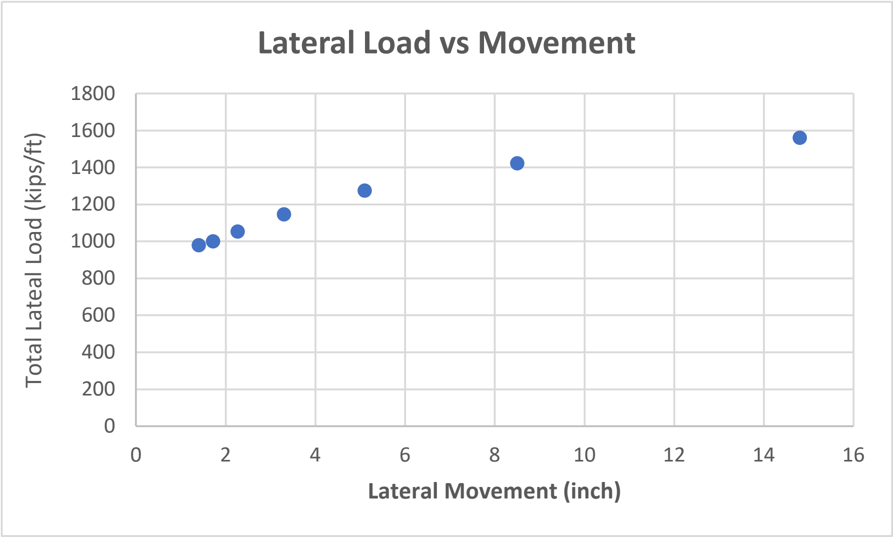
FE Model Calibration for Soil Springs





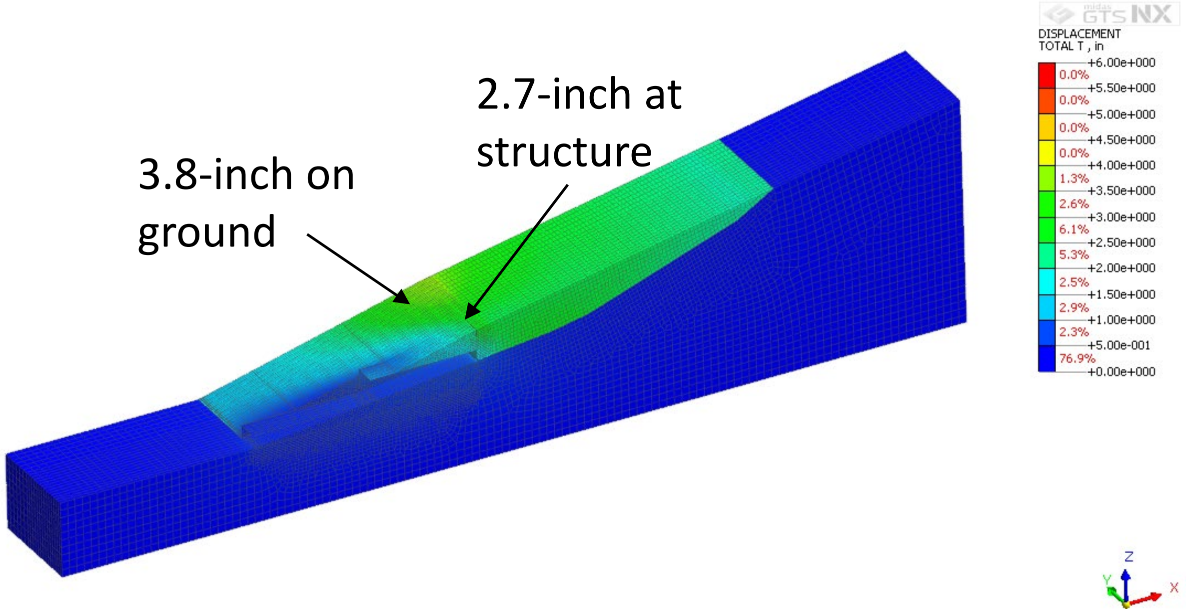
Case	Soil Spring (k/ft)	Reaction Force (kips/ft)	Lateral Movement (inch)	Resultant Lateral Pressure (psi)
1	15000	980	1.4	80
2	7500	1000	1.7	81
3	3750	1054	2.3	86
4	1875.0	1147	3.3	93
5	937.5	1276	5.1	104
6	468.8	1423	8.5	115
7	234.4	1561	14.8	127
8	187.5	1599	N/A	N/A

Calibrated Soil Spring

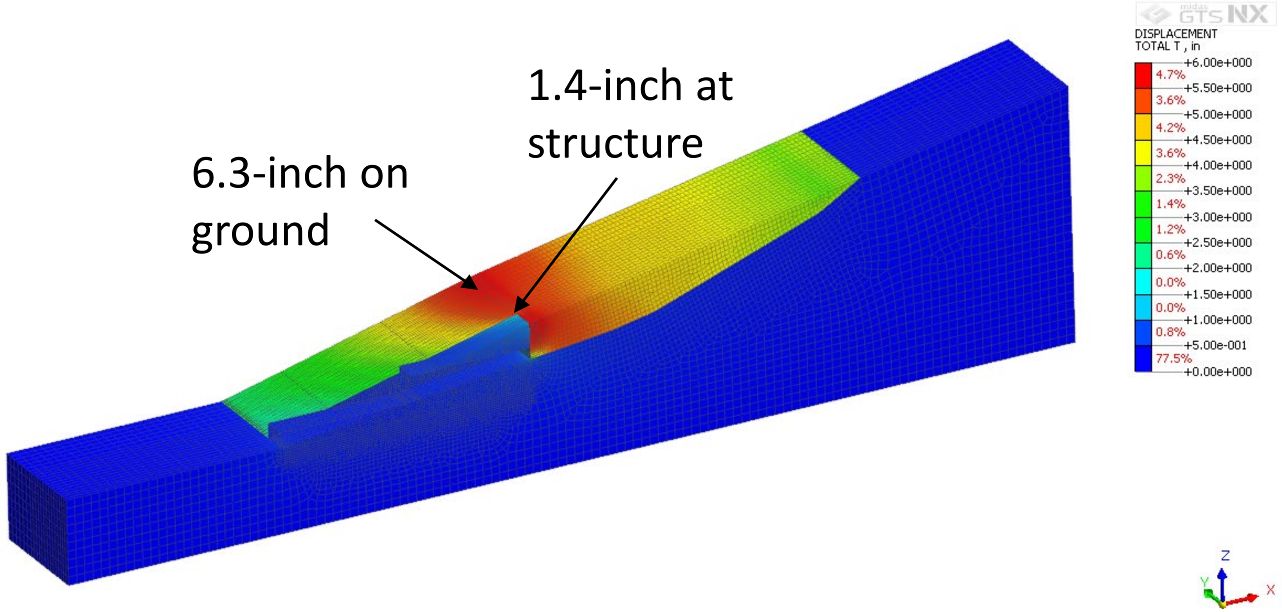


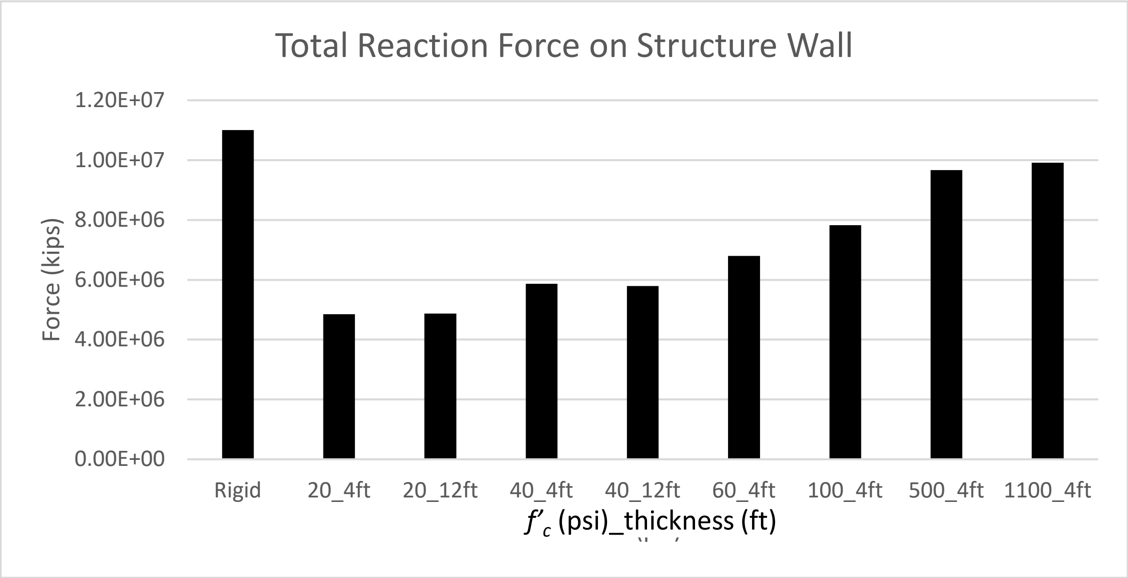
Lateral Displacement of Earth Flow

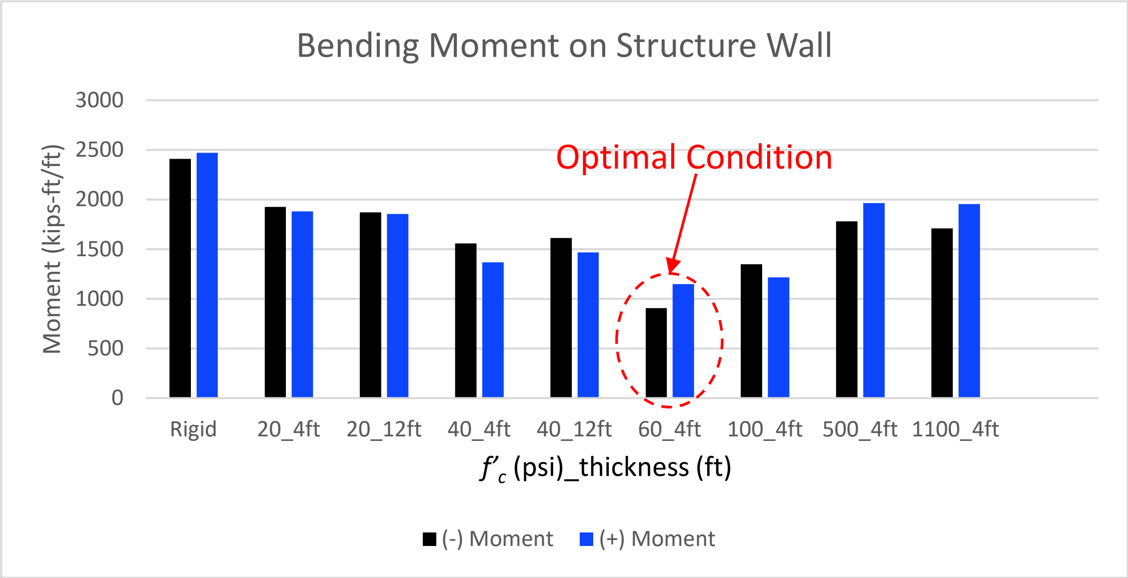
(a) Without EDAS



(b) With EDAS and 30 psi capacity concrete

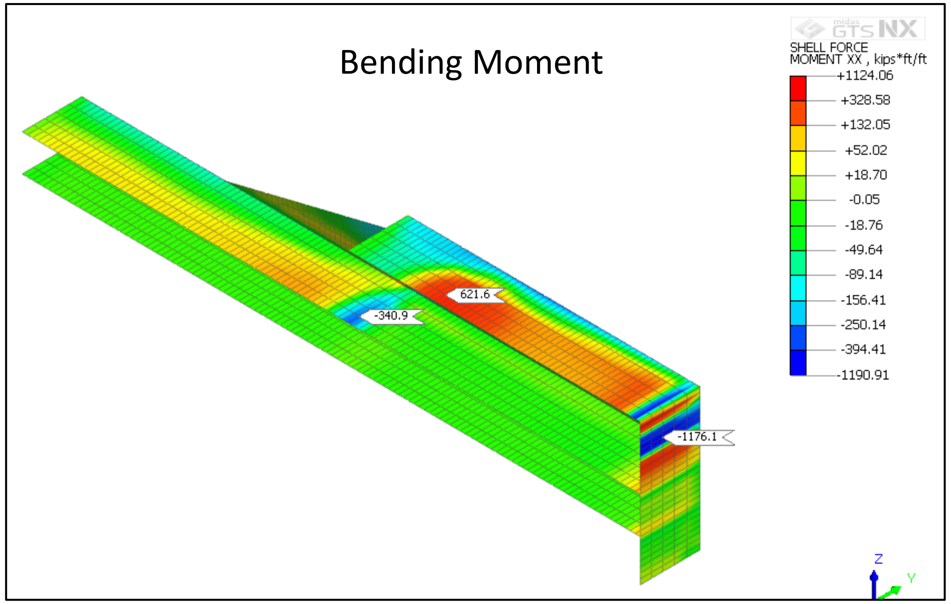






Findings:

- The total reaction forces acting on the front wall increase with the stiffer crushable concrete.
- After interactions between structure and crushable concrete, the induced internal bending moments on the structure members are not linear relations with the stiffness of crushable concrete, and shows an optimal condition, which should be associated with the interaction between the stiffness of crushable concrete and structure's global stiffness.



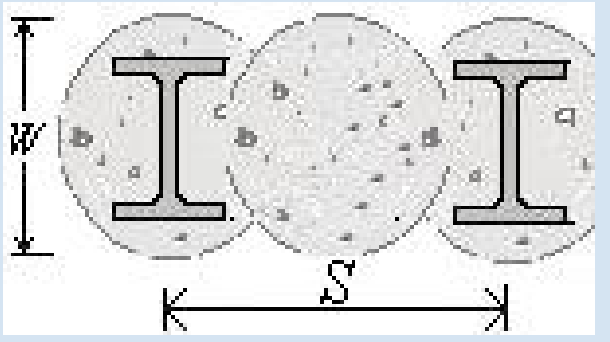
2.3 South Portal - Retaining Wall Design

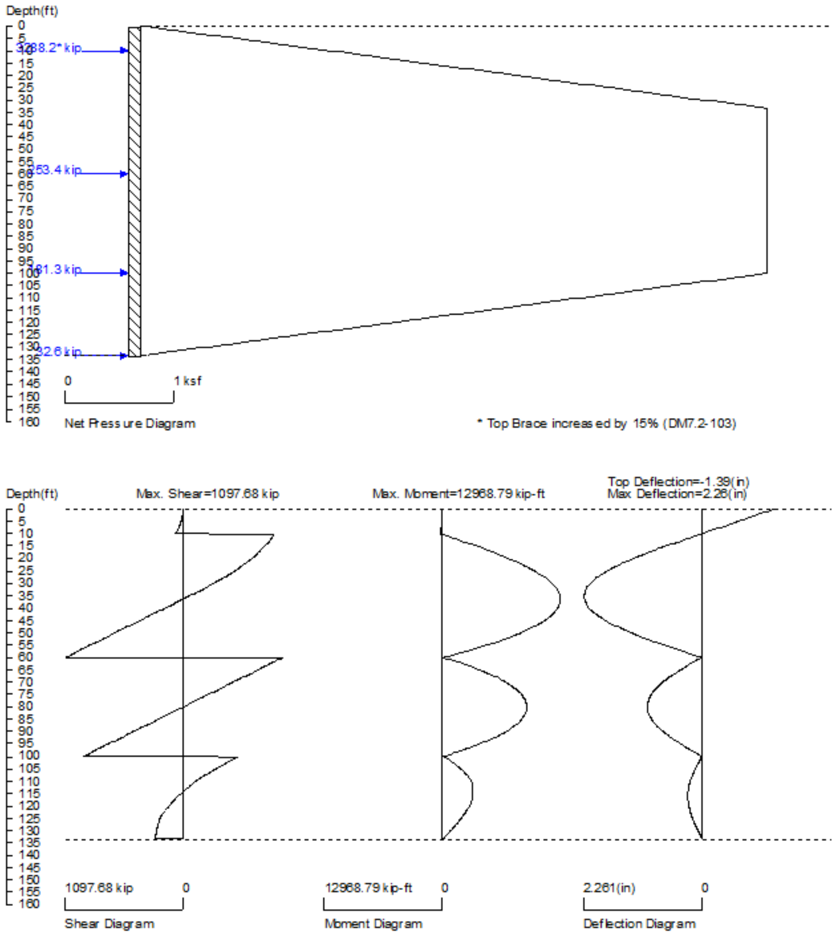
Construction Methodology and Design

- **Top-Down Excavation**
- **Initial configuration inputs of structural members of retaining wall**
- **Using SHORING SUITE to preliminary design of Secant Pile Wall**
- **Using ACI-318 to preliminary design of bracing members**

SHORING SUITE to Preliminary Design of Slabs

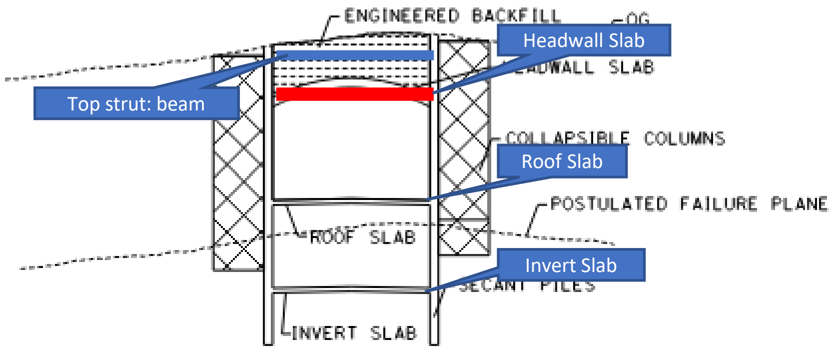
Diameter of Scant Pile W=5ft
C-C Spacing of Secant Piles, S=8ft
Core Beam: S44x290
Embedment Length =10ft (at EL540')





PRESSURE, SHEAR, MOMENT, AND DEFLECTION DIAGRAMS

ACI-318 Flexure Design of Slab



Span of headwall slab = 67.5ft

Overburden, q = 60’x0.12kcf = 7.2ksf

$M_{max} = qL^2/12 = 7.2*76.5^2/12 = 7.2ksf$

Assuming height of slab, h=6ft

Effective depth of slab:

d = 6*12-3-1.5*1.41-5/8 = 66.26in

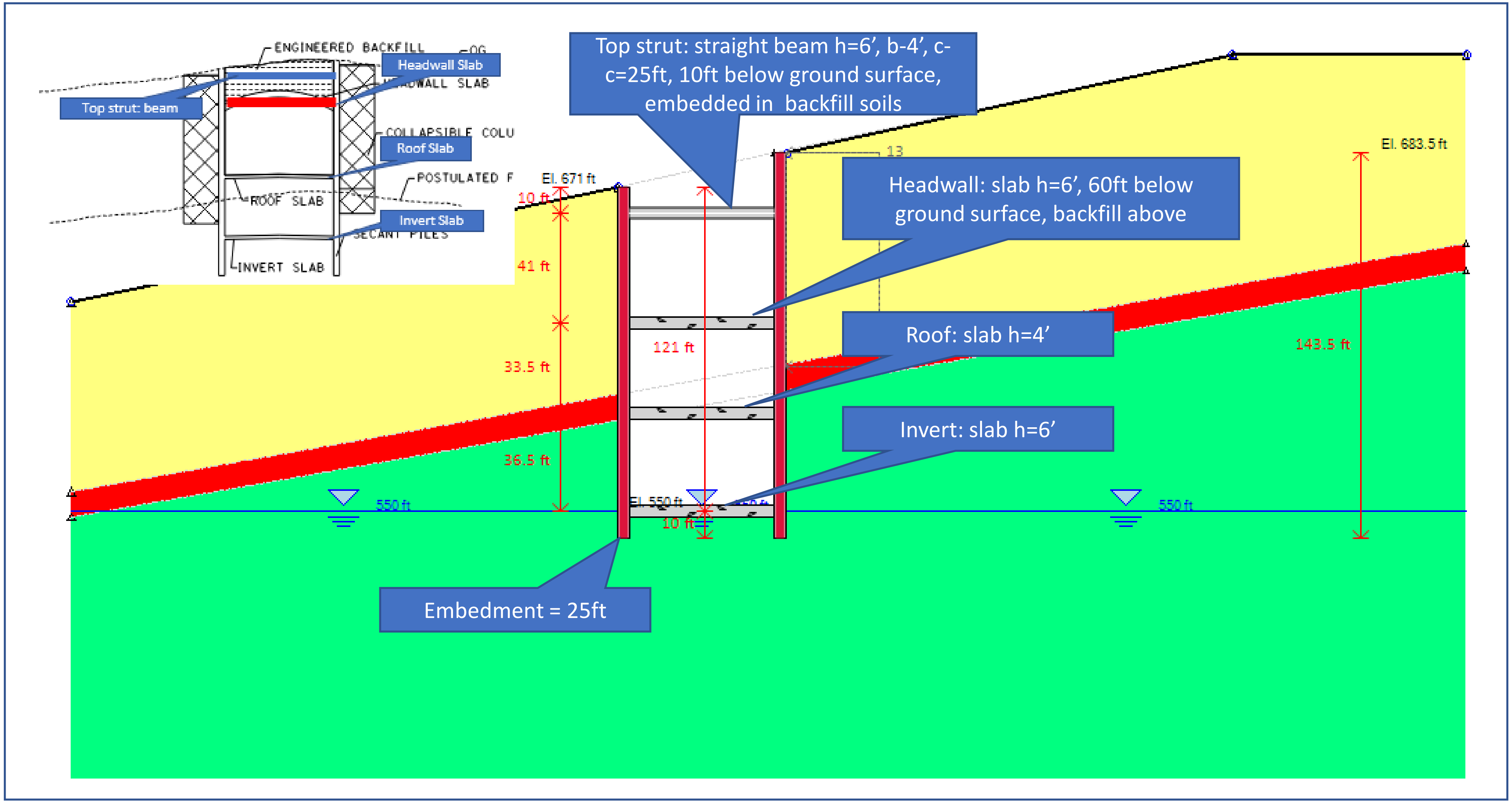
So, Height of headwall slab, suggested as 6ft

ACI 318 Flexure Design of Slab

Only Yellow Cells To Be Input

Input		
Factored Bending Moment, M_u	32805	k-in
Effective Depth of Beam, d	64.85	in
Strength Reduction Factor, ϕ	0.9	-
Compressive stress of Conrete, f_c	5	ksi
Yield Stress of Steel, f_y	60	ksi
Intermediate Parameters		
Nominal Bending Moment, $M_n = M_u / \phi$	36,450.00	k-in
Nominal Strength Coefficent of Resistance, $R_n = M_n / [bd^2]$	0.7222646	-
Result		
Reinforcement Ratio, $\rho = 0.85f_c/f_y*[1-(1-2*R_n/(0.85*f_c))]^{0.5}$	0.0132832	-
Area of Tension Steel, $A_s = \rho bd$	10.3	in^2
Check		
Depth Ratio of Rectangular Stress Block to Neutral Axis, β	0.8	
Min Reinforcement Ratio, ρ_{min}	0.002	
Max Reinforcement Ratio, ρ_{max}	0.0251531	
Reinforcement Limits, $\rho_{min} <= \rho <= \rho_{max}$	OK	

Design of Retaining Walls



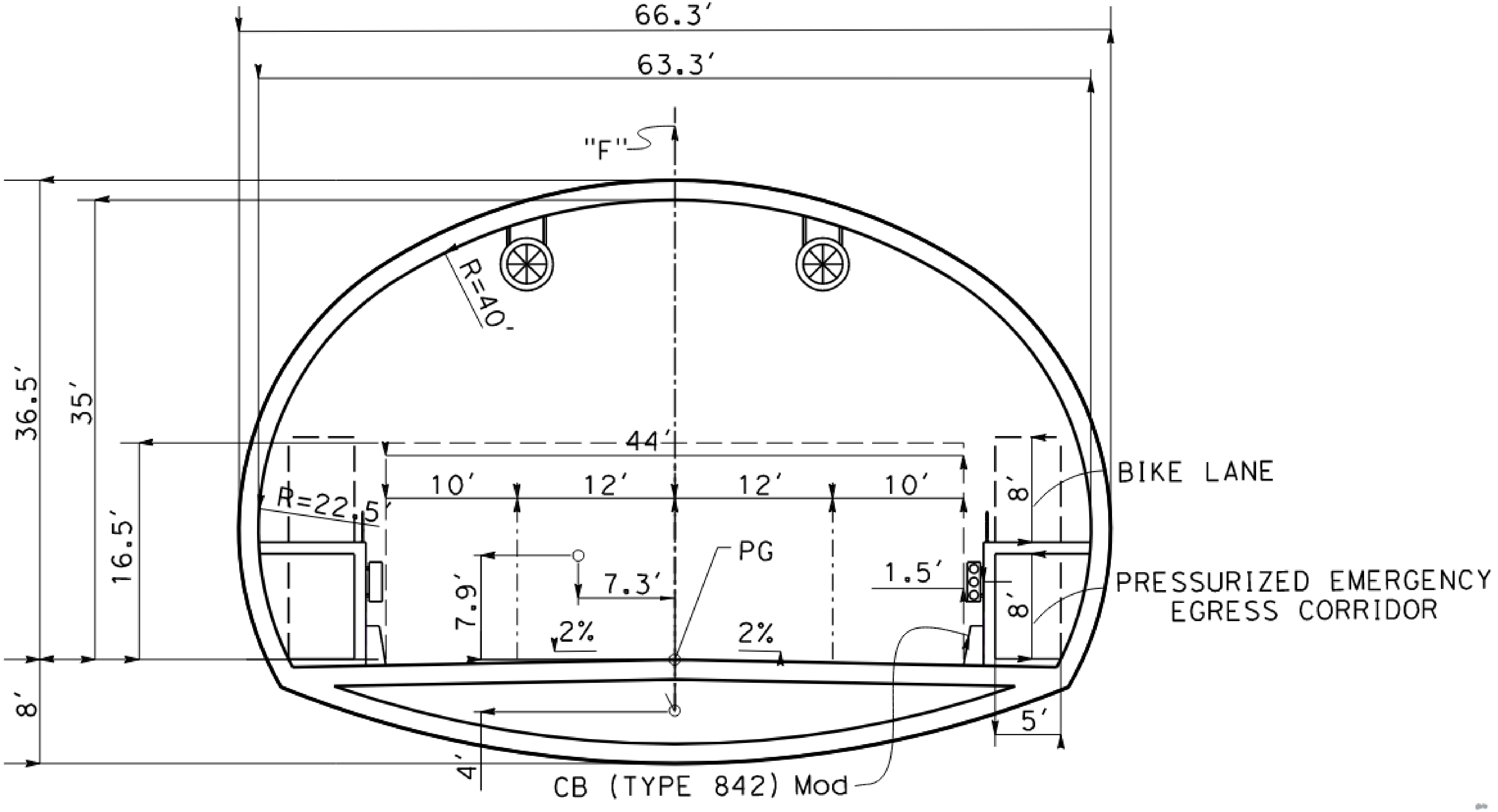
3. Tunnel - Seismic Ovaling Analysis and P-M Diagrams

(Seismic Condition Governs Lining Design)

Three Tunnel Cases Studied:

- **Deep 1.5 ft thick tunnel lining**
- **Deep 2.0 ft thick tunnel lining**
- **Shallow 2.0 ft thick tunnel lining**

Tunnel Section



Seismic-Induced Free-Field Ground Shear Deformation

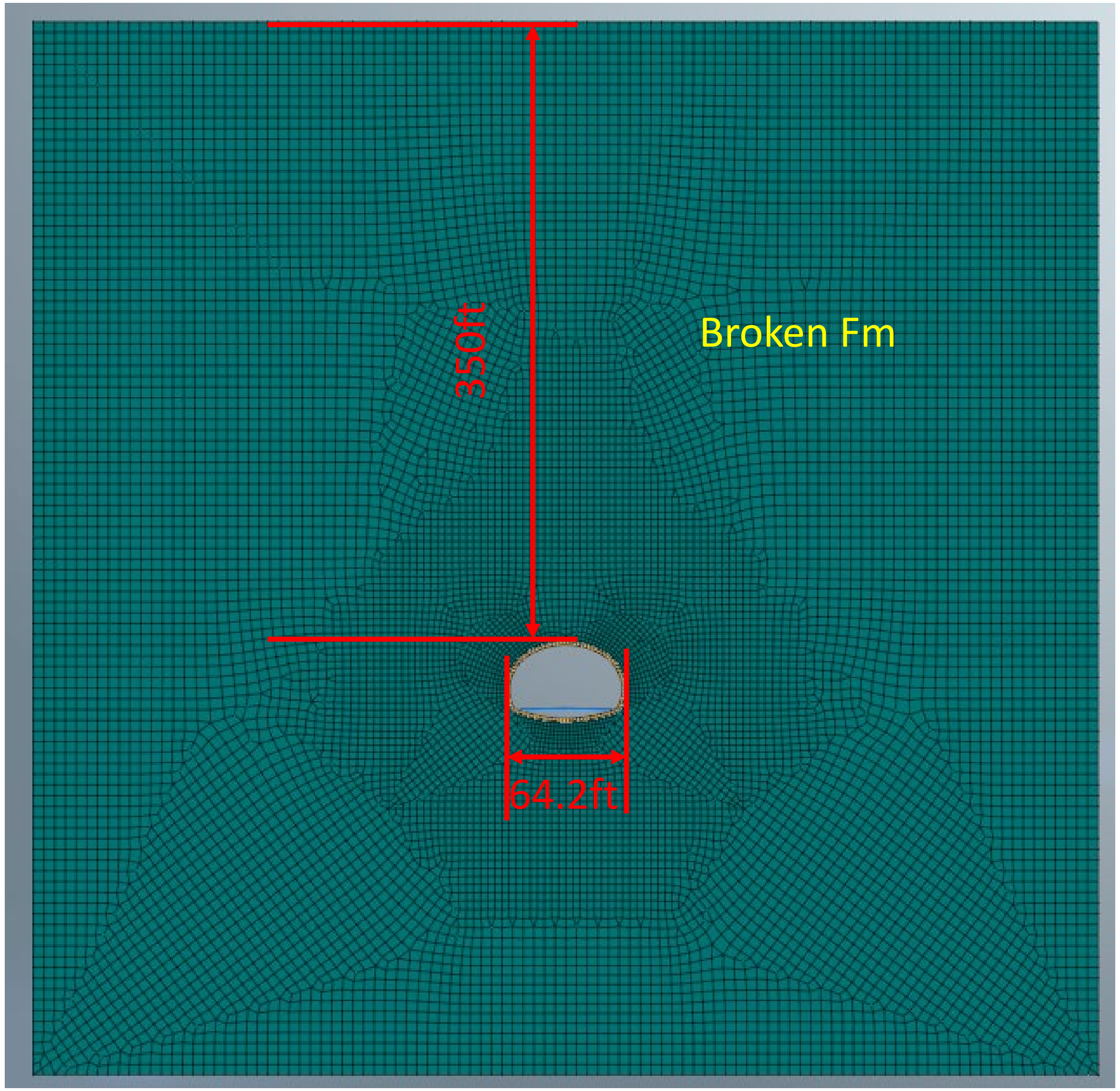
Deep Tunnel

Shear Wave Velocity of Ground, Vs	3000	ft/s
Max shear modulus, G_{max}	39,162	ksf
Ratio of shear modulus, G/G_{max}	0.61	-
Strain-compatible shear modulus, G_m	23888.78	ksf
Equivalent unit weight of ground, γ	140	pcf
Poisson's ratio of ground, ν_m	0.4	-
Peak ground acceleration, PGA	0.7	g
Ground depth, H	350	ft
Tunnel diameter, D	56	ft
Effective shear wave progation velocity, C_{se}	2343.1	ft/sec
Strain-compatible elastic modulus, E_m	66888.584	ksf
Mass density of the ground, ρ	4.35133	pcf/g
Depth from ground surface tunnel invert, z = H+d	406	ft
Vetical Stress at tunnel invert, σ_v	56.8	ksf
Depth dependent stress reduction factor, R_d	0.5	-
Peak shear stress, τ_{max}	19.9	ksf
Peak free field shear strain, γ_{max} using PGA	8.33E-04	-
Non-perforated diameteric change, $\delta D_{free\text{-}field}/D$, i.e., ground stiff = liner stiff	4.16E-04	-
Perforated Diameteric change, $dD_{free\text{-}field}/D$, i.e., ground stiff >>liner stiff	9.99E-04	-
Outer Diameter, OD	56	ft
Poisson's ration of liner, ν_l	0.2	-
Thinkness of liner, t_l	1.5	ft
Number of segment, n	8	-
Inner Diameter, ID	53	ft
Nominal radius of the tunnel liner, R_l	27.25	ft
Moment of inertia of liner per unit width of tunnel along the tunnel axial, I	0.281	ft^4
Equivelent moment of inertia, I_e	0.070	ft^4
Equivalent thickness liner using Muri Wood Eq., t_e	0.945	ft
Elastic modulus of liner, E_l	580393	ksf
Compressibility Ratio, C	11.4	-
Flexiblity Ratio, F	3790.4	-
Lining response coefficient, K_1	0.00095	-
Lining response coefficient, K_2	0.54	-
Liner Diameteric strain, $\Delta D_{max}/D$	9.99E-04	-
Maximum thrust, T_{max}	290.6	k/ft
Maximum bending moment, M_{max}	4.7	k-ft/ft
Resuliting bending moment induced maximum fiber strain, ε_m	5.41E-05	-
The Axial force induced strain, ε_T	5.30E-04	-

Shallow Tunnel

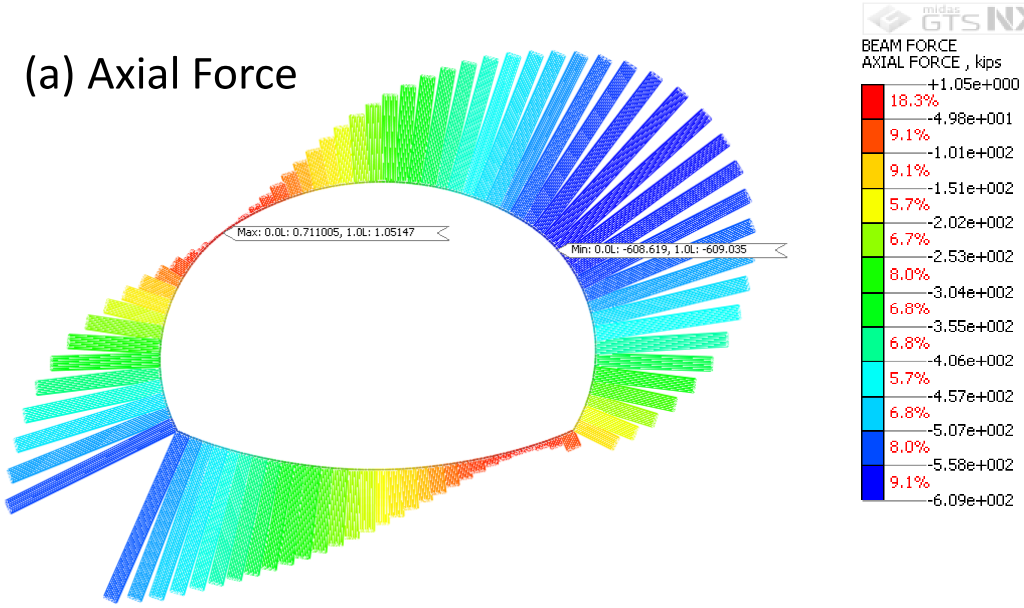
Shear Wave Velocity of Ground, Vs	3000	ft/s
Max shear modulus, G_{max}	39,162	ksf
Ratio of shear modulus, G/G_{max}	0.45	-
Strain-compatible shear modulus, G_m	17622.871	ksf
Equivalent unit weight of ground, γ	140	pcf
Poisson's ratio of ground, ν_m	0.4	-
Peak ground acceleration, PGA	0.7	g
Ground depth, H	150	ft
Tunnel diameter, D	56	ft
Effective shear wave progation velocity, C_{se}	2012.5	ft/sec
Strain-compatible elastic modulus, E_m	49344	ksf
Mass density of the ground, ρ	4.35133	pcf/g
Depth from ground surface tunnel invert, z = H+d	206	ft
Vetical Stress at tunnel invert, σ_v	28.8	ksf
Depth dependent stress reduction factor, R_d	0.5	-
Peak shear stress, τ_{max}	10.1	ksf
Peak free field shear strain, γ_{max} using PGA	5.73E-04	-
Non-perforated diameteric change, $\delta D_{free\text{-}field}/D$, i.e., ground stiff = liner stiff	2.86E-04	-
Perforated Diameteric change, $dD_{free\text{-}field}/D$, i.e., ground stiff >>liner stiff	6.87E-04	-
Outer Diameter, OD	56	ft
Poisson's ration of liner, ν_l	0.2	-
Thinkness of liner, t_l	1.5	ft
Number of segment, n	8	-
Inner Diameter, ID	53	ft
Nominal radius of the tunnel liner, R_l	27.25	ft
Moment of inertia of liner per unit width of tunnel along the tunnel axial, I	0.281	ft^4
Equivelent moment of inertia, I_e	0.070	ft^4
Equivalent thickness liner using Muri Wood Eq., t_e	0.945	ft
Elastic modulus of liner, E_l	580393	ksf
Compressibility Ratio, C	8.4	-
Flexiblity Ratio, F	2796.2	-
Lining response coefficient, K_1	0.00129	-
Lining response coefficient, K_2	0.62	-
Liner Diameteric strain, $\Delta D_{max}/D$	6.87E-04	-
Maximum thrust, T_{max}	170.2	k/ft
Maximum bending moment, M_{max}	3.2	k-ft/ft
Resuliting bending moment induced maximum fiber strain, ε_m	3.72E-05	-
The Axial force induced strain, ε_T	3.10E-04	-

MIDAS Model
Case 1, Deep Large Diameter Single Tunnel

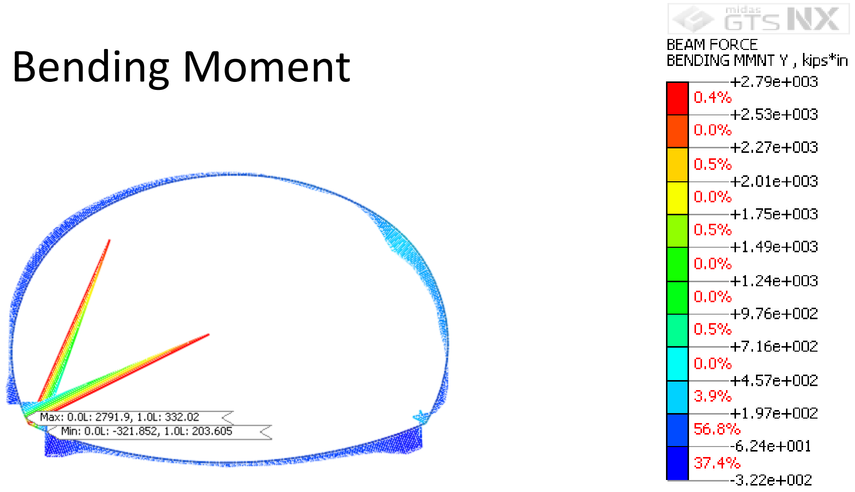


Results of MIDAS Analysis
Case 1, Deep Large Diameter Single Tunnel

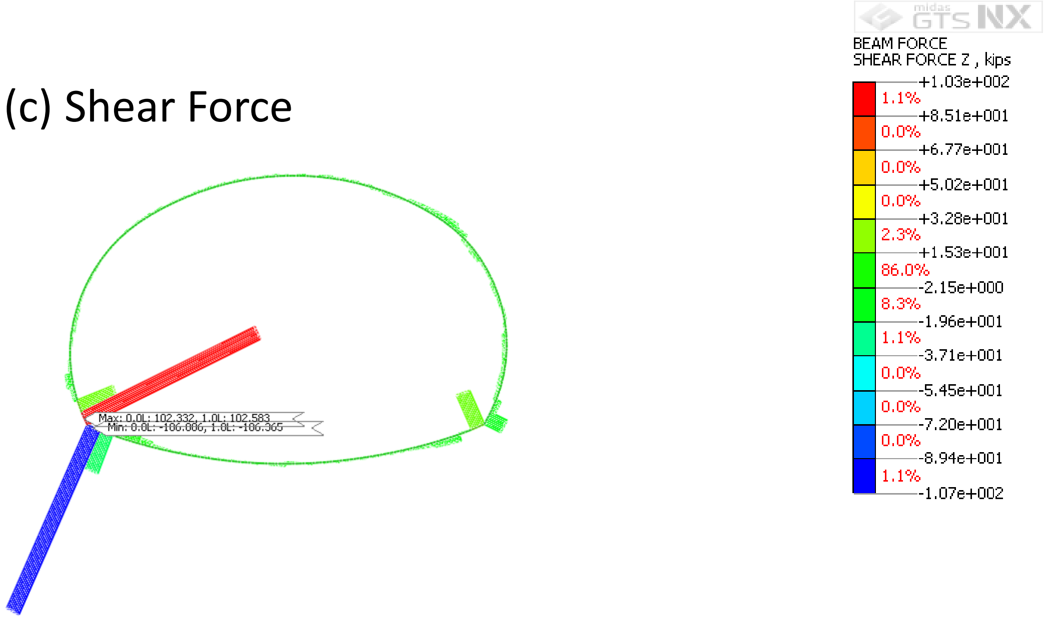
(a) Axial Force



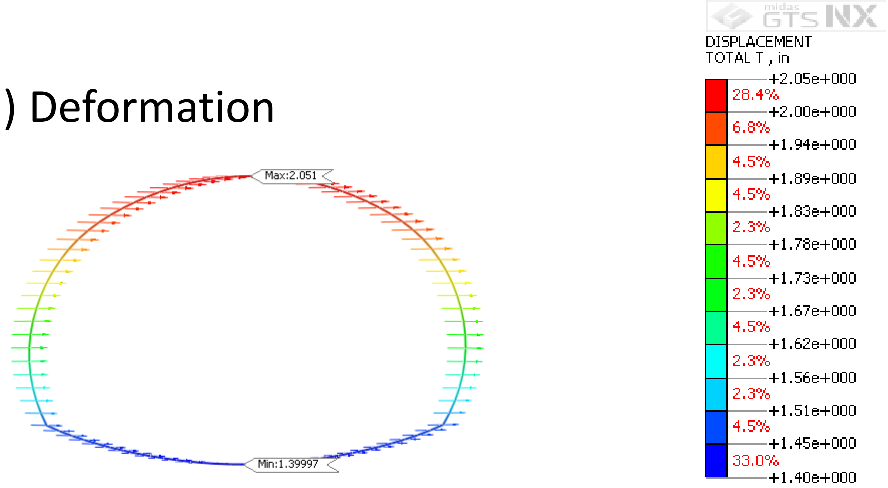
(b) Bending Moment



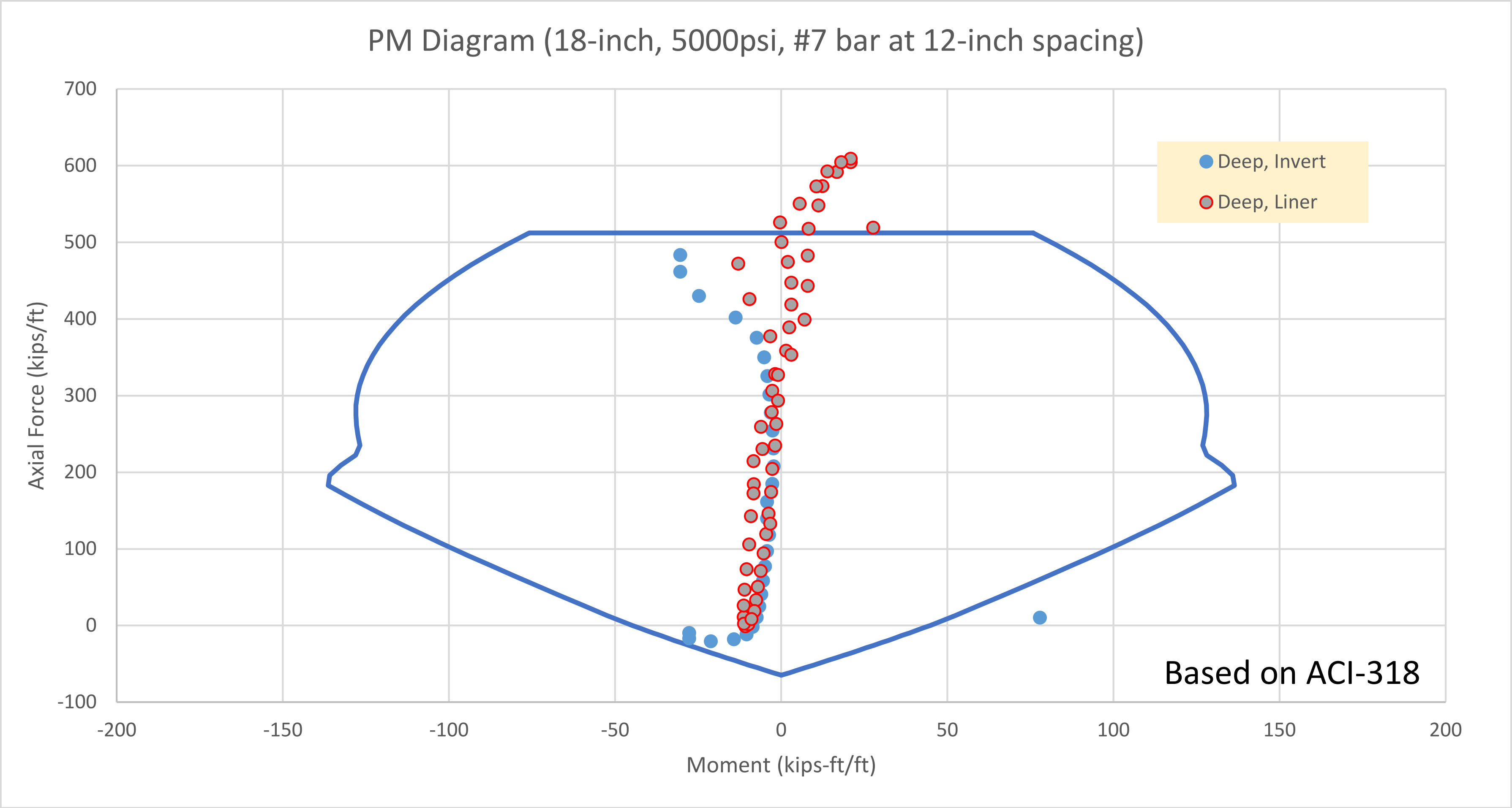
(c) Shear Force



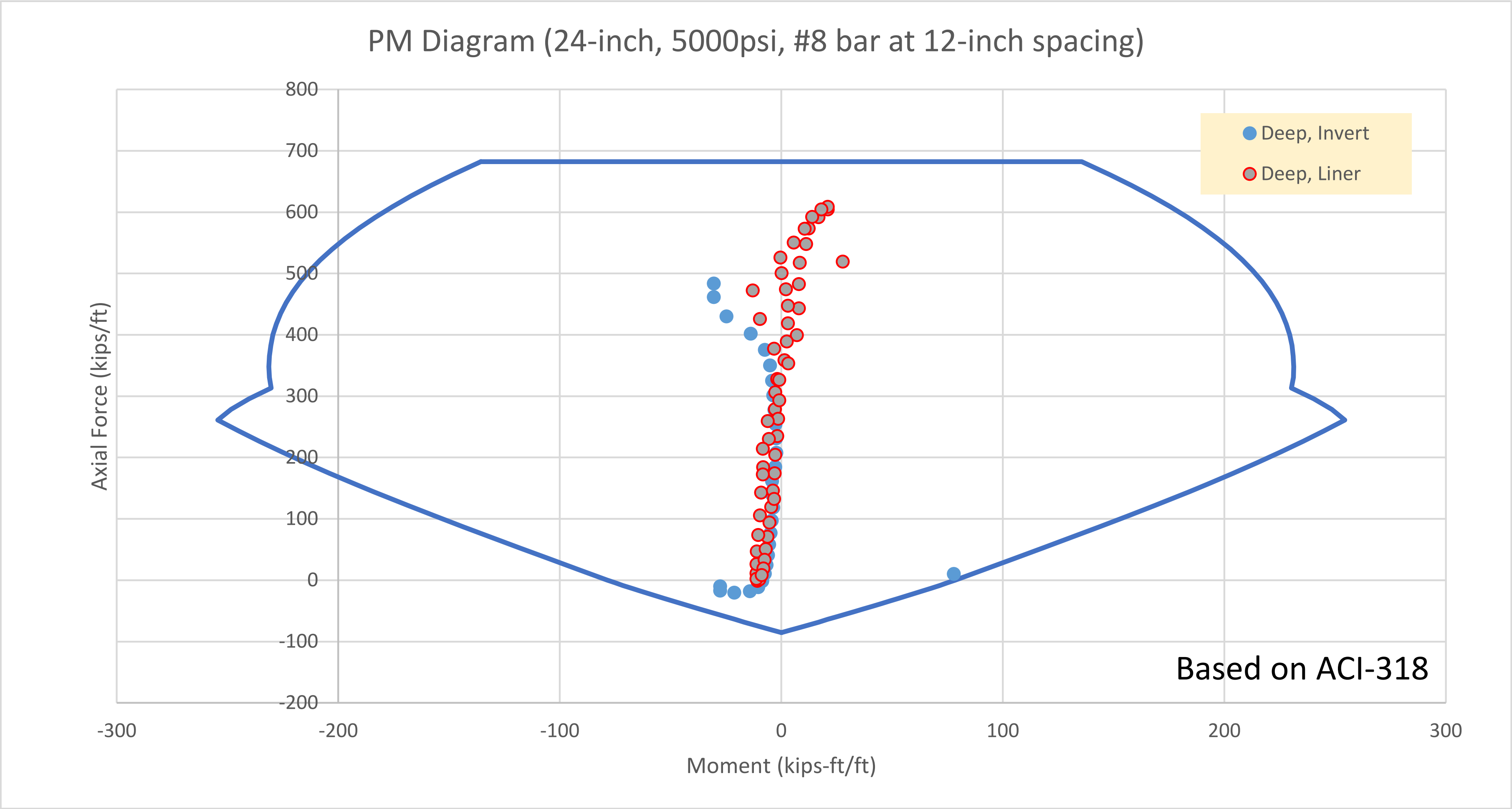
(d) Deformation



MIDAS Model
Case 1, Deep Large Diameter Single Tunnel, Thickness of Liner = 18in

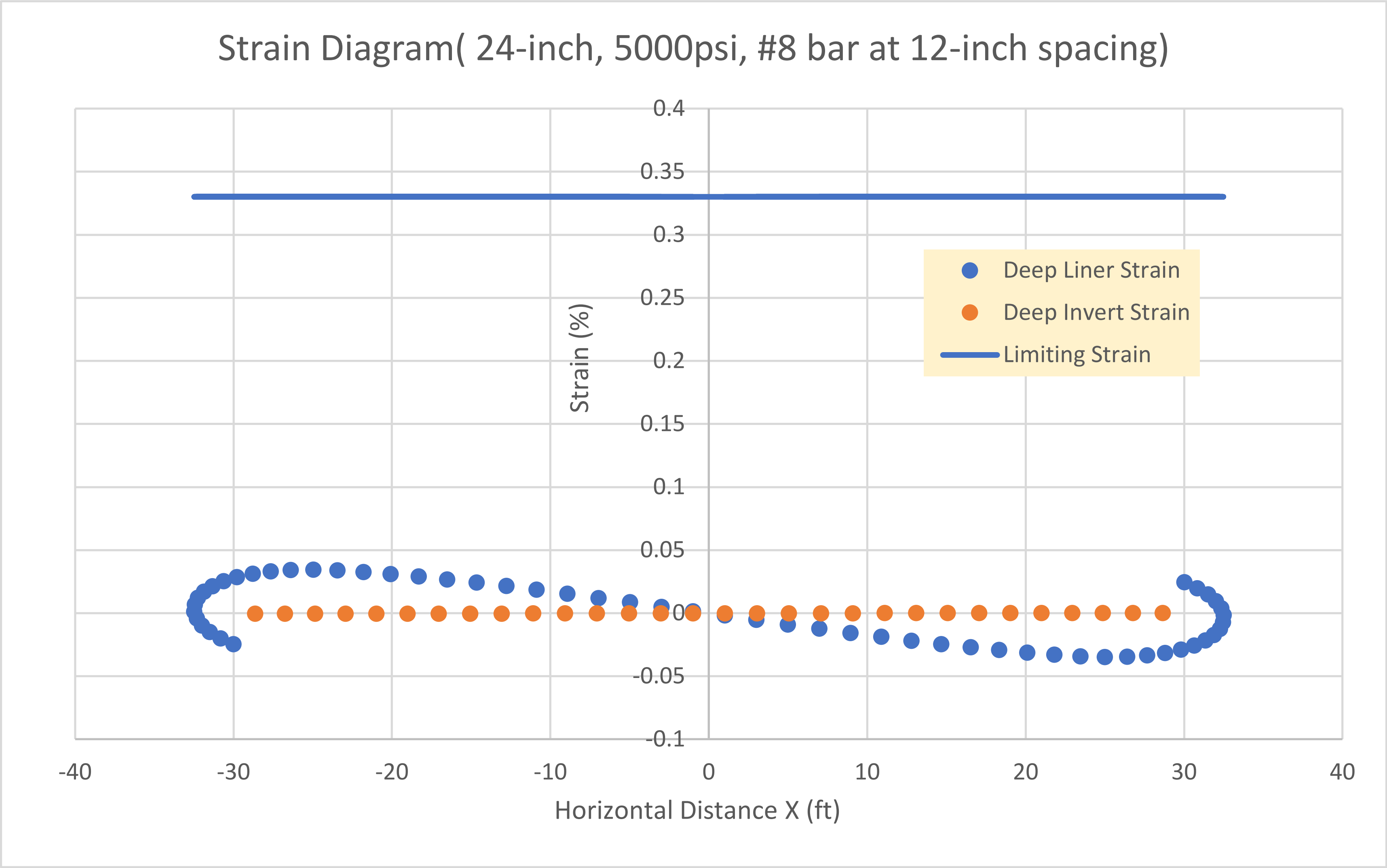


MIDAS Model
Case 2, Deep Large Diameter Single Tunnel, Thickness of Liner = 24in

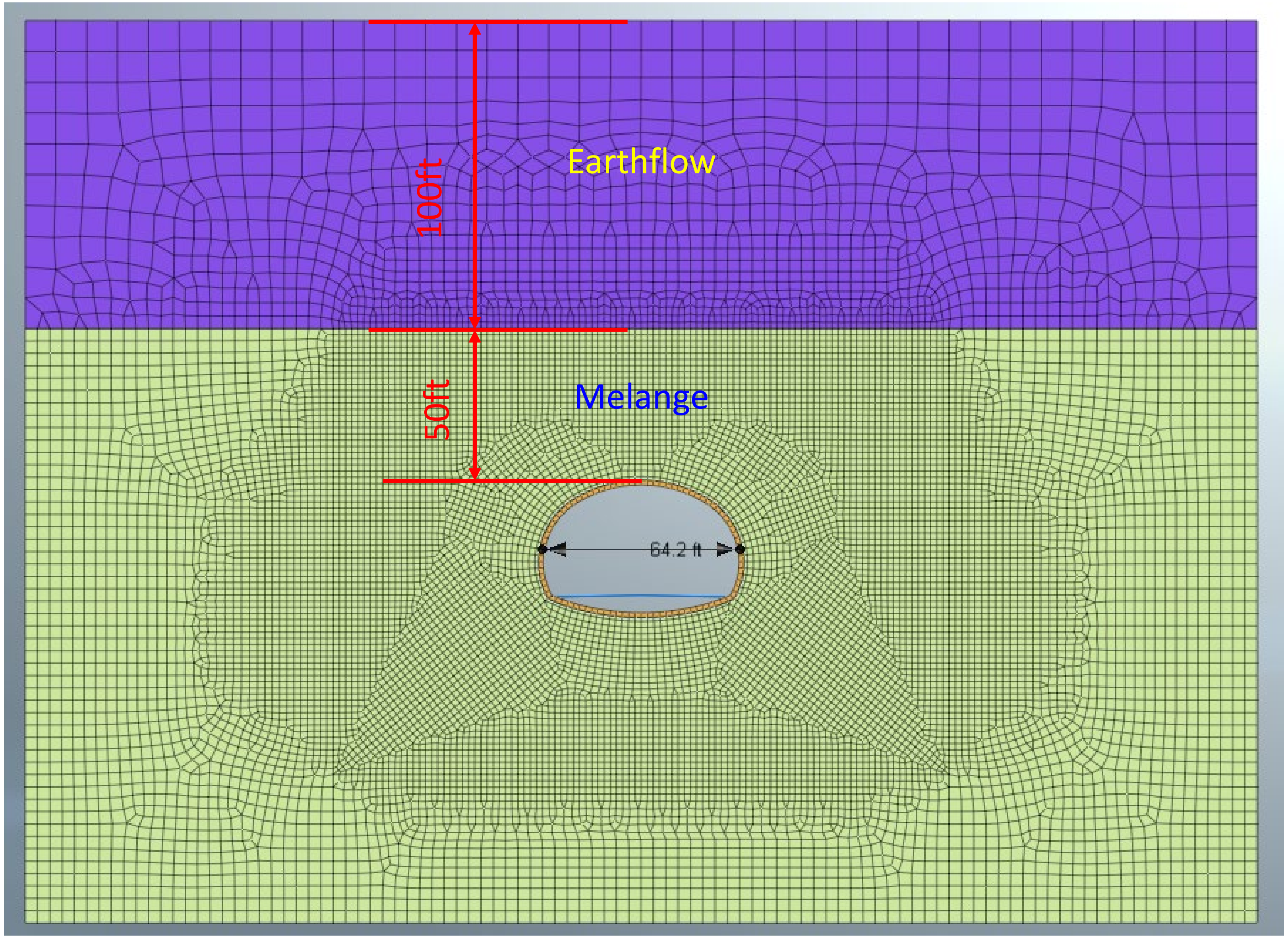


Strain Diagram

Case 2, Deep Large Diameter Single Tunnel, Thickness of Liner = 24in

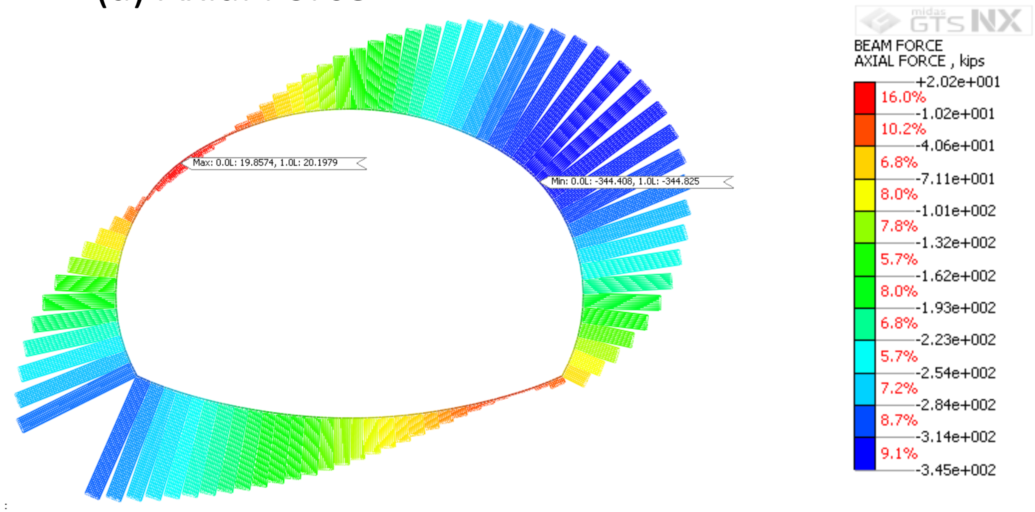


Results of MIDAS Analysis
Case 3, Shallow large Diameter Single Tunnel, Thickness of Liner = 24in

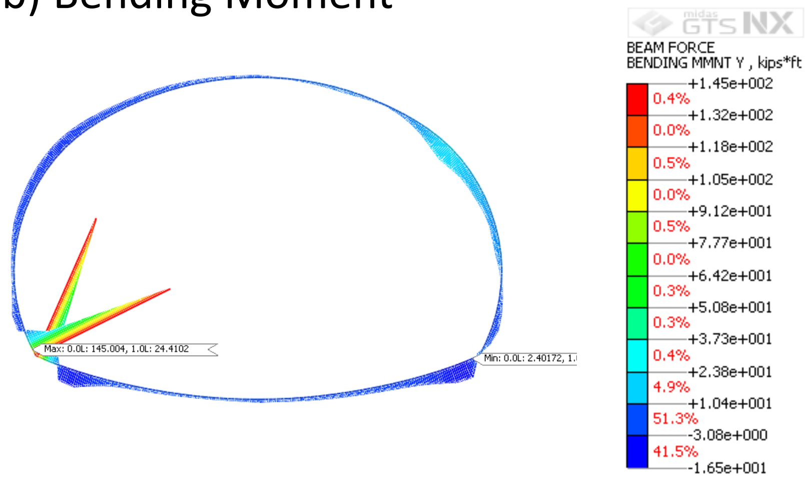


MIDAS Model
Case 3, Shallow large Diameter Single Tunnel

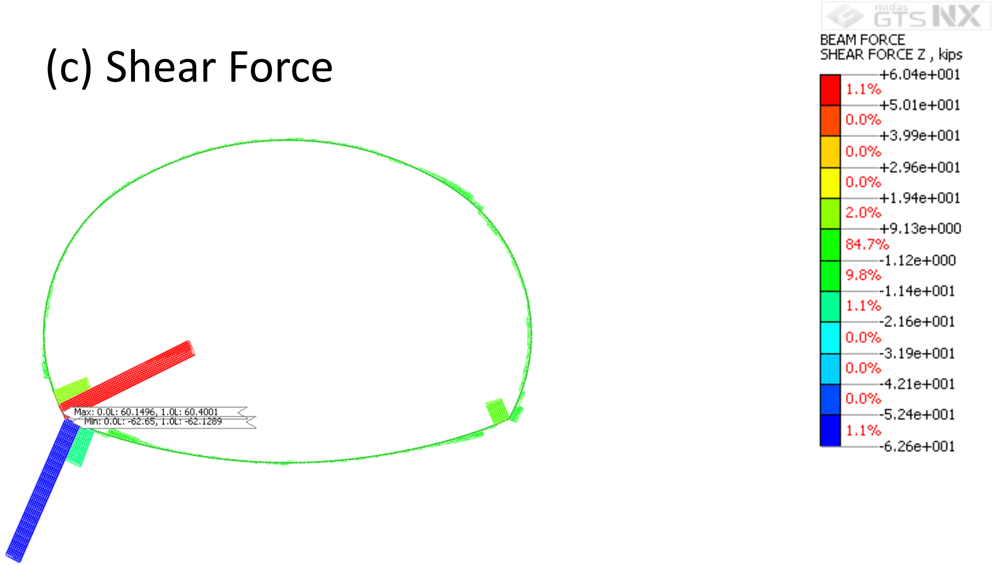
(a) Axial Force



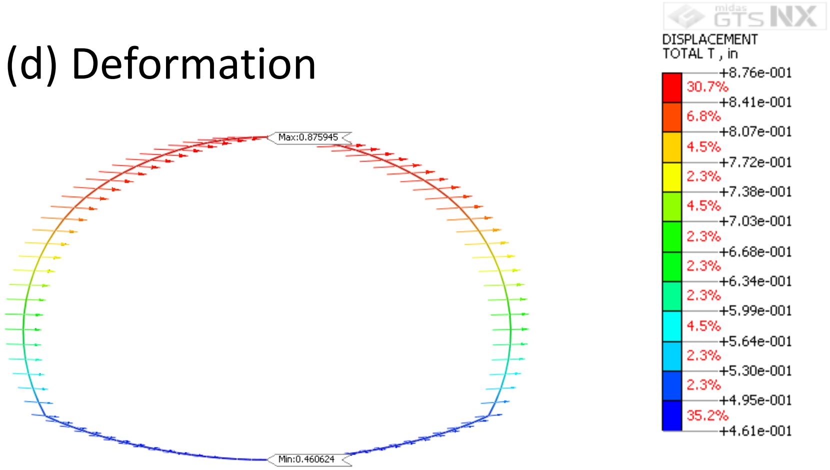
(b) Bending Moment



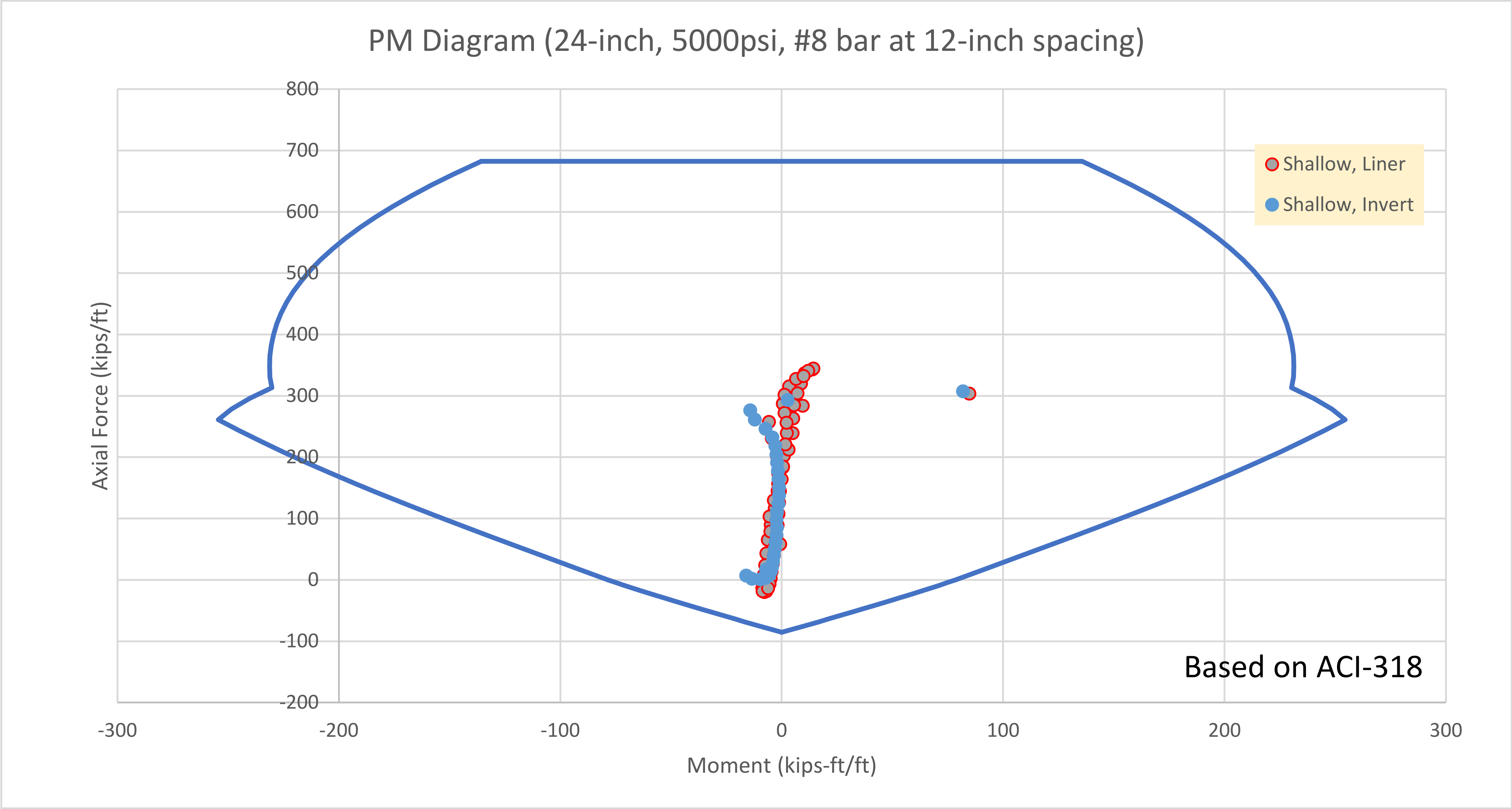
(c) Shear Force



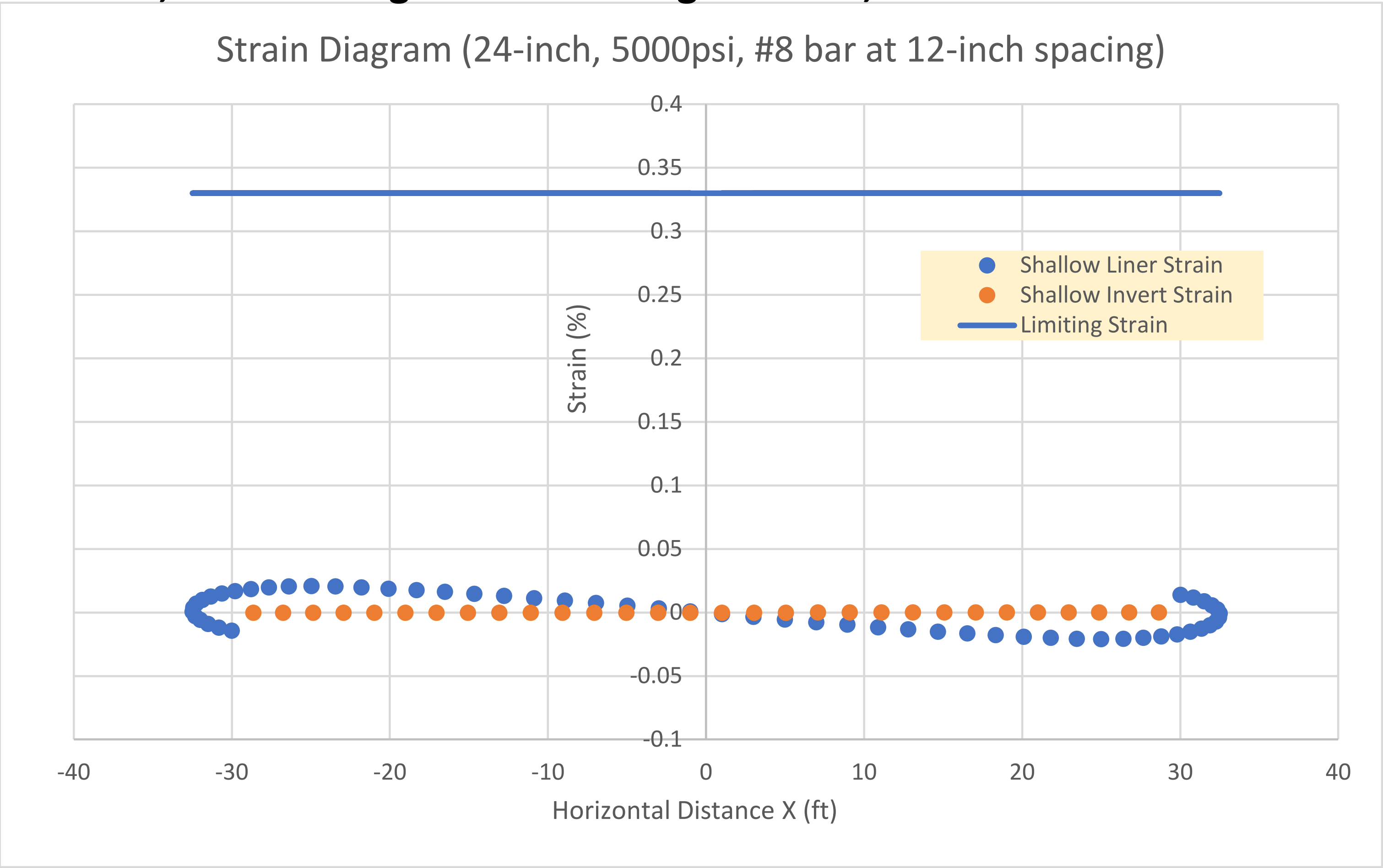
(d) Deformation



PM Diagram
Case 3, Shallow large Diameter Single Tunnel, Thickness of Liner = 24in



Strain Diagram
Case 3, Shallow Large Diameter Single Tunnel, Thickness of Liner = 24in



4. Findings

- The proposed collapsible concrete columns are effective in moderating loads on the approach structure at south portal.
- For the large SEM tunnel, lining thickness of 24 inches is compatible with projected seismic strains.
- North tunnel portal to be located east of the existing slide surfaces and in a rock cut area.