SUPPLEMENTAL
GEOTECHNICAL DATA REPORT
PROPOSED CHANGES TO MINING PLAN
SAN RAFAEL ROCK QUARRY
MARIN COUNTY, CALIFORNIA

VOLUME 1 OF 2

SUBMITTED

TO

DUTRA MATERIALS

SAN RAFAEL, CALIFORNIA

PREPARED

BY

ENGEO INCORPORATED

PROJECT NO. 6261.1.003.01

APRIL 11, 2005

ENGEO INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
6261.1.003.01

April 11, 2005

Mr. Brian Peer
Dutra Materials
1000 Point San Pedro Road
San Rafael, CA 94901

Subject: San Rafael Quarry
San Rafael, California

SUPPLEMENTAL GEOTECHNICAL DATA REPORT

Dear Mr. Peer:

With your authorization, we have prepared this supplemental data report for the San Rafael Quarry in Marin County, California.

The purpose of this report is to provide supplemental data to support the conclusions of the September 9, 2004, geotechnical report prepared by ENGEO. This supplemental data was provided at the request of personnel at the State Office of Mining Reclamation (OMR). The most significant component of this supplemental report was the modification of previously submitted cross-sections and related stability analyses to reflect the pit configuration submitted with ARP 04. The pit configuration used in our previous report was a preliminary mining plan that was modified just prior to submittal of our report. Analyses presented in this report indicate that substitution of the ARP 04 plan has resulted in increased stability in all cases. As the attached data and analyses demonstrate, the conclusions and recommendations of the 2004 ENGEO report have been confirmed by this supplemental report. In order to create a report that could be reviewed as an independent document, much of the previous report was excerpted and included in this supplemental report.

We are pleased to provide our services to you on this project and look forward to consulting further with you and your design team.

Very truly yours,

ENGEO INCORPORATED

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VOLUME 2

EXECUTIVE SUMMARY

Purpose and Scope

The purpose of the September 9, 2004, ENGEO Incorporated geotechnical report was to provide a preliminary analysis of rock slope stability at the San Rafael Rock Quarry (SRRQ) based on the quarry's plan to mine the resource to -350'MSL (with a sump to -400'MSL) as reflected in ARP04 and working with exposed conditions in the existing quarry pit and on available data on subsurface conditions around the pit. This supplemental report is presented to provide the additional data and analyses requested by the State Office of Mining Reclamation (OMR) to test and further explore the conclusions in the original report. Based on the preliminary analyses, we provide here our geotechnical opinions regarding:

- The feasibility of SRRQs plan to mine to -350"MSL.
- The feasibility, in geotechnical terms, of the post-reclamation Second Uses contained in ARP82 and ARP04 as submitted.

The scope of work for this study included the following:

- Review of previously published maps, consultant reports and boring logs describing geological and geotechnical characteristics of the subject site and nearby vicinity.
- Review of materials testing data from the quarry and from State studies in similar rock conditions for the San Francisco Bay Bridge seismic retrofit project.
- Review of published information on intact rock and discontinuity shear strength.
- Review of several sets of aerial photographs flown for mining quantity estimates.
- Geologic mapping of exposed quarry faces.
- Measurement of joint orientation, surface characteristics, and infill material at 47 locations around the pit.

- Review of previous topographic maps of the site area.
- Interviews with quarry personnel, including the current manager, Mr. Peer, and the previous manager, Mr. Woodbury.
- Logging of 17 air-percussion borings to depths of approximately 120 feet on and near the north quarry brow and south of the central portion of the south quarry brow.
- Analysis of the geological and geotechnical data.
- Supplemental analyses recommended by OMR, including construction of additional cross sections, additional slope stability analyses and evaluation of seismic amplification effects.
- Preparation of this report summarizing our findings and recommendations.

This report was prepared for the exclusive use of Dutra Materials and its design team consultants. In the event that any changes are made in the character, design, or layout of the reclamation plan, the conclusions and recommendations contained in this report should be reviewed by ENGEO Incorporated to determine whether modifications to the report are necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO Incorporated.

Conclusions

- Analyses presented in this report show that the pit configuration in ARP 04 has resulted in increased stability in all cases. As the attached data and analyses demonstrate, the conclusions and recommendations of the September 9, 2004, ENGEO report have been confirmed by this supplemental report. Therefore, based on the results of our field mapping, subsurface exploration, data review and analyses, it appears that ARP 04 can be implemented as proposed.
- The deposit currently mined at the quarry consists of hard graywacke sandstone of the Franciscan Assemblage. In the quarry pit area, the mineable deposit continues to a depth of at least 550 feet below mean sea level. Mineable sandstone is also present at depth below the South Hill area, below a weathered rock deposit. The geotechnical characteristics of the rock mass, including discontinuities, groundwater conditions and intact rock strength were evaluated based on the best available information from site-specific geologic mapping, previous detailed

rock core logs, published Bay Area geologic and geotechnical information using analysis methods appropriate for rock slope engineering. Rock slope stability was evaluated for the existing pit, the maximum proposed mining depth and for the post-mining, flooded pit condition. The proposed deepened pit was analyzed for static conditions and for earthquake conditions (pseudo-static loading, topographic seismic amplification effects and Newmark deformation analysis).

- The calculated static and pseudo-static factors of safety for quarry pit slopes are within generally acceptable limits for mining at the proposed final pit elevation of -350 feet msl and after the pit is flooded for adjacent residential, marina and commercial uses. Topographic seismic amplification effects are not expected to adversely affect post-mining reclamation and second use plans, assuming that the recommendations of this report are adhered to. Seismic slope deformation calculated for quarry pit slopes based on a Newmark analysis is estimated to be negligible.

- Prior to the final design of proposed improvements, the findings of this report should be re-evaluated based on exposed post-mining conditions within the pit. The location of proposed habitable structures and critical facilities such as lifeline roads and utilities with respect to the top finished pit reclamation slopes should be based on detailed post-mining studies.

- Conservative geotechnical assumptions were used throughout the analyses presented in this report, with the intent to provide a geologic model with very conservative factors of safety for planning purposes. Recommended future studies will include geologic mapping as new exposures are created during mining, additional rock shear strength testing, and monitoring of groundwater levels. Based on additional data, it may be possible to justify higher rock mass shear strengths and locally steeper slope inclinations if appropriate.

Recommendations

A. Supplemental Geotechnical Pit Observations

- The pit exposures should be observed and evaluated as mining proceeds by a qualified engineering geologist and/or mining engineer. The purpose of observation during mining would be to identify possible adverse rock structure as excavations proceed, so that the quarry operations can avoid undesirable slope failures in critical improvements such as access ramps or quarry brow improvements. Since much of the proposed quarrying area is currently not exposed, supplemental observations will be carried out to adequately address geologic conditions as they are revealed by mining.

- As a minimum, a thorough re-evaluation of excavated slopes should be performed near the conclusion of the mining operations so that the proposed post-reclamation conversion to Second Uses contained in ARP82 and ARP04 can be re-evaluated based on revealed conditions.

B. Groundwater Monitoring

- We recommend that piezometers be installed around the existing pit margin during the balance of the mining process to measure the elevation of the existing piezometric surface or surfaces and to confirm assumptions about pore water pressures made for this analysis as monitoring proceeds.

- The actual configuration of the piezometer array should be determined based on the final proposed pit configuration and on proposed planning of quarry operations to allow optimum placement of instruments and to avoid conflicts with future operations.

C. Slope Monitoring

- We recommend that a network of survey monitoring points be established at selected locations around the pit brow and on selected benches. The survey net should be monitored monthly during periods of active mining or at more frequent intervals if appropriate to monitor suspected areas of movement.

D. Recommended Future Studies

- We recommend that the quarry pit slope stability characterization presented in this report be supplemented at the conclusion of mining, and prior to conversion to the Second Uses contained in ARP04, with a comprehensive re-evaluation of quarry slope stability based on the results of on-site geotechnical pit observations made during mining, groundwater monitoring, slope monitoring, and a program of laboratory testing of on-site materials. An appropriate testing program should, as a minimum, include unconfined compression tests, triaxial testing, and direct shear tests of joint surfaces.

- At the conclusion of mining activities, a qualified engineering geologist or mining engineer should prepare a revised geologic map of the pit and provide supplemental recommendations for implementation of the proposed reclamation plan. If required, supplemental rock slope engineering recommendations should be provided to maintain acceptable factors of safety for proposed land uses.

- The preliminary descriptions presented in this report of subsurface soil conditions outside the pit area should be confirmed with a comprehensive geotechnical site exploration. We anticipate that such an exploration will include geotechnical borings and sampling of site soils, laboratory testing and analyses of collected data focused on providing design-level recommendations to mitigate the geotechnical development constraints identified in this study.

- The design-level geotechnical report should include detailed and site-specific recommendations for grading, mitigation of compressible and liquefiable soils, slope stability analyses, recommendations for appropriate foundations for residential and commercial structures and geotechnical recommendations for construction of underground utilities and surface drainage improvements. In concert with the final design of site grading plans, ENGEO typically provides the client with a detailed corrective grading plan showing the locations of keyways slope stabilization earthwork, subdrains, and the limits and depth of removals intended to mitigate geotechnical condition such as compressible or unsuitable soils. Corrective grading plans for this site would also include depictions of the recommended limits of geotechnical remediation measures such as surcharge fills, wick drains, structural earth retention and ground improvement.

- All recommended geotechnical remediation measures should be coordinated with the project civil engineer so that they can be properly incorporated in the final grading plans.

E. Quarry Slope Design

- Within the quarry pit, the average (toe to top) slope inclination should not exceed 60 degrees for a maximum vertical height of 350 feet, as depicted on Figure 15.

- Minimum 30-foot-wide safety benches should be constructed at maximum 90-foot vertical intervals.

- In general, the inclination of inter-bench faces should be maintained at less than 75 degrees where possible. The recommended safety bench spacing and width are depicted on Figure 14. Locally, inter-bench face inclinations will be influenced by splitting along pre-existing rock discontinuities, but overhanging faces should be avoided wherever possible. The bench and slope angle recommendations presented here have been incorporated into ARP 04.

- The quarry access ramp placement required to deepen the quarry has been configured to minimize excavation at the south face and create a buttressing effect to the slopes at the south side of the quarry.

- Quarry pit design should consider the potential effect of large-scale horizontal curvature of pit walls on slope stability. In general, convex-inward horizontal curves in quarry slopes should be avoided. Concave inward slopes offer some degree of increased confinement by "arching" of the rock mass between discontinuities, and effectively decrease the area of free face available for kinematically possible failure geometries. Convex-inward slopes can actually contribute to potential instability, since lateral confinement is reduced and the area of the kinematically-available free face is effectively increased. The concave-inward slopes contained in the plan to deepen the quarry and reflected in ARP 04 is more appropriate for stability than the configuration shown in ARP82.

F. Slope Stability Mitigation Options for Mining

- The periodic geotechnical inspections recommended above should include evaluation of mining faces for potentially unstable blocks. Localized face failures are an expected part of surface mining, and the location and potential size of unstable blocks can be evaluated during periodic inspections as mining proceeds. If it appears that a critical facility such as the access ramp could be threatened by a potential block failure, the geotechnical engineer could recommend appropriate corrective action such as the installation of rock bolts, or local modification of mining excavations to increase stability.

- The large-scale stability of the quarry walls should be periodically evaluated by the geotechnical engineer based on the results of monitoring of slope performance, groundwater levels, and geotechnical inspection of mining exposures. If unacceptable slope performance is detected, it will be possible to implement several possible mitigation measures as described below. The actual recommended mitigation measures should be based on site-specific evaluations and should be implemented based on an appropriate evaluation of risk and cost.

- Mitigation measures should be employed if adverse groundwater conditions are encountered (unacceptably high pore pressures or excessive seepage, etc.) Mitigation measures could include horizontal drains, extraction wells, slurry walls, etc.

- If unacceptable levels of mining-concurrent slope deformation are encountered, mining activities can be modified to improve stability. At the quarry brow, stockpiles of products, quarry waste piles or areas of overburden can be excavated and moved to reduce driving forces. In the pit, bench configurations can be modified by "stepping out" or increasing bench width, effective flattening the mining slope angle.

- At the south quarry brow, it is anticipated that the final slopes will locally expose quarry fills and areas of native soils and weathered rock. The anticipated extent of soils and weaker materials in the proposed face is presented in Figure 13. Figure 14 presents options for

mitigation, including construction of a sheet pile wall or an engineered fill buttress. Both options would allow the quarry limits depicted in SRRQs mining plan to be preserved.

G. Location of Improvements Near Quarry Pit Slopes

- Prior to the final design of proposed improvements, the findings of this report should be re-evaluated based on exposed post-mining conditions within the pit.

- The location of Second Use structures and critical facilities such as lifeline roads and utilities with respect to the top finished pit reclamation slopes should be based on the results of the recommended detailed post-mining studies.

INTRODUCTION

Purpose and Scope

The purpose of the ENGEO report of September 9, 2004, was to provide a preliminary analysis of rock slope stability at the San Rafael Rock Quarry (SRRQ) based on the quarry's plan to mine the resource to -350'MSL (with a sump to -400'MSL) consistent with ARP04 and working with exposed conditions in the existing quarry pit and on available data on subsurface conditions around the pit. This supplemental report is presented to provide the additional data and analyses requested by the State Office of Mining Reclamation (OMR) to test and further explore the conclusions in the original report. Based on the preliminary analyses, we have provided herein our geotechnical opinions regarding:

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Site Location and Description

The San Rafael Quarry is located northeast of San Rafael at Point San Pedro in Marin County, California, as shown on Figure 1. The quarry property consists of approximately 285 acres bounded on the northwest by San Pedro Road, and on the south and east by the waters of San Francisco Bay. We understand that the property has been used for surface mining purposes since the late 1800s. The original mined resource was meta-shale used for brick production. Beginning in the late 1970s, the ridge referred to on United States Geological Survey (USGS) topographic maps as San Pedro Hill was mined for sandstone aggregate. Since that time, surface mining and associated material

handling operations have encompassed most of the former San Pedro Hill, and the existing pit extends to an elevation of approximately 200 feet below mean sea level (msl).

Proposed Land Use after Mining is Complete

The Amended Reclamation Plan of 1982 (ARP 82), and now ARP04, anticipates that, after mining is complete, the Main Quarry Bowl would be flooded to create a harbor and areas around the brow of the existing quarry pit and in the Northwest, Northeast and Southwest Quadrants of the site would be contoured to accommodate residential and commercial Second Uses including streets and underground improvements as well as surface drainage and shoreline protection improvements. It is also anticipated that the existing hill (the south Hill) southwest of the pit will also be re-contoured to create a series of slopes and level areas to accommodate post-mining Second Uses.

PREVIOUS STUDIES

The quarry was studied between the 1970s and 1990s by several consultants, including Woodward-Clyde Consultants (WCC) in 1981, Geomatrix Consultants (GC) in 1989, and Golder Associates (GA) in 1991. The WCC, GC and GA studies included auger borings, deep mud-rotary probes and cored borings across much of the quarry property at locations shown on Figure 5. A table summarizing the previous drilling history is presented in Appendix C. Borings from the WCC, GC and GA reports are attached as Appendix C.

Wyllie & Norrish Rock Engineers Inc. (WN) has prepared a report dated October 2004 that commented on quarry reserves and quarry pit design. The WN pit design presents the basic pit geometry used for the slope stability models presented in this report.

REGIONAL GEOLOGY AND SEISMICITY

Regional Geology

The geology of Marin County has been mapped on a regional scale by Blake, et al. (1974) and Blake, et al. (2000). The mapping of Blake, et al. (2000) is presented on Figure 3. The bedrock at San Pedro Point is mapped as Franciscan Assemblage sandstone of Late Cretaceous age. The Franciscan Assemblage is divided into several fault-bounded terrains with differing rock types and metamorphic grades. According to Blake, et al. (2000), the Franciscan Assemblage in the San Rafael-Novato area is part of the Novato Quarry Terrane, a sequence of interbedded sandstone and meta-shale with local massive sandstone channel deposits. The rocks of the Novato Quarry Terrane are slightly metamorphosed to prehnite-pumpellyite grade (Blake, et al., 2000).

Faulting and Seismicity

No active faults are known to pass through the property, according to Blake, et al. (2000) and State of California Earthquake Fault Hazard Zone maps. The known active faults closest to the site are the Hayward Fault located approximately 4.7 miles (7.6 km) to the east and the Rodgers Creek Fault, located approximately 9.4 miles (15.1km) to the northeast. The San Andreas Fault is located approximately 13 miles (20.9 km) to the southwest. Figure 4 shows the approximate location of Quaternary faults and significant historic earthquakes mapped within the San Francisco Bay Region.

The regional seismicity of the Bay Area was recently evaluated by the Working Group on Northern California Earthquake Probabilities (WGEP, 2003). The Working Group periodically attempts to summarize seismic risk in the Bay Area by presenting probabilities of $6.7M_w$ or greater earthquakes on active Bay Area faults for a 30-year return interval; the most recent summary gives a 62 percent aggregate probability for the entire Bay Area. The Working Group has assigned contributing probabilities to the above aggregate of 27 percent and 21 percent for the Hayward-Rodgers Creek and San Andreas Fault systems, respectively.

SITE GEOLOGIC CONDITIONS

Pre-Mining Conditions

As described above, the topography of the quarry site has been extensively modified since surface mining began in the late 1800s. Review of the 1895 15-minute San Francisco Quadrangle shows that mining at the site has lowered the ridge from an original elevation of 200 to 300 feet above sea level. The 1895 map also shows a natural shoreline that appears to have included both steep rocky bluffs and gradually sloping beach areas. At the south quarry brow, the 1895 shoreline varies from approximately 400 to 500 feet from the present pit to as close as 200 feet. The area where the old shoreline is nearest the existing pit was a local embayment. The shape of the shoreline suggests that the embayment was possibly bordered by a sandy beach. North of the shore, the ground depicted on the 1895 map rose steeply to the former ridge. Along the line of the current pit edge, the elevations on the 1895 map ranged from approximately 25 to over 200 feet above sea level. The lowest 1895 elevations along the pit edge occurred at the axis of a topographic swale. This area also coincides with the most weathered portion of the rock mass exposed in the pit, and with the thickest deposits of un-consolidated materials encountered in borings, as described below. The approximate location of the old shoreline is depicted on the Cross Sections, Figures 10 and 11.

Summary of Existing Subsurface Soil Information, South Shoreline

The soil deposits around the quarry have been penetrated by a number of borings done for previous studies and by air-percussion borings done for the 2004 ENGEO report and for this report. The previous subsurface studies at the site were generally focused on evaluating rock quality. The previous borings were drilled with auger, mud-rotary, and air-percussion drill rigs. Some of the previous mud-rotary borings by GA and GC were sampled in the soil intervals with Standard Penetration Test (SPT) drive samplers, and therefore, were logged on the basis of both

recovered samples and on cuttings returned by the drilling fluid. We are not aware of any published laboratory test results from the sampled GA or GC borings. Other borings by GA, GC, WCC and ENGEO were not sampled in the soil layers above the bedrock, and were logged in those intervals from cuttings returned to the surface. Some of the air-percussion borings did not return cuttings within the surface in soil layers, and therefore, were logged on the basis of the drilling resistance encountered by the downhole hammer. Groundwater depth in auger and air-percussion borings was noted if encountered. Groundwater depth could not be accurately determined in mud-rotary borings since they were drilled with water as the circulation fluid and were not instrumented. The "depth to mud" noted on some mud-rotary logs is an approximation of the groundwater elevation at the time of drilling and is noted below. The logs of borings and a summary of existing borings describing drilling method elevation and total depth are presented in Appendix C. A summary of subsurface conditions encountered in the borings is given below.

TABLE 1
SUBSURFACE SOIL CONDITIONS AT SOUT PROPERTY MARGIN

BORING	DRILLING METHOD	BORING ELEVATION (FT)	BASE OF FILL (FT)	FILL CLASSIFICATION (USC)	BASE OF NATIVE SOIL/TOP OF ROCK (FT)	NATIVE SOIL CLASSIFICATION (USC)	NATIVE SOIL DESCRIPTION ****	GROUNDWATER DEPTH (FT)
P-1	Auger	18	3	GW	3	--	--	***
P-2	Auger	13	25	GW	37	CL	Qbm	***
P-3	Auger	9	12	SW	12	--	--	***
P-4	Auger	15	5	SW	5	--	--	***
B-1	Auger/SPT	5	22	SW/GW	50	CH	Qbm	6*
B-2	Auger/SPT	8	5	SC	18	CL/SC	Qbm	8.5*
B-3	Auger/SPT	22	18?	GW	18	--	--	**
B-4	Auger/SPT	25	3	GP	3	--	--	**
B-5	Mud Rotary	19	51	SW/GW	76	CL	Qbm	18.9*
B-6	Mud Rotary	23	56.5	SW/GW	65.5	CL	Qbm	20.6*
B-7	Mud Rotary	20	29?	SW/GW	36	CL/GC	Qbm?	***
U-2	Mud Rotary	12	51	SW/GW	51	--	--	***
U-3A	Mud Rotary	12	42	SW/GW	72	CL?	Qbm?	***
WB-1	Mud Rotary	16.5	12	ML/SM	12	--	--	***

BORING	DRILLING METHOD	BORING ELEVATION (FT)	BASE OF FILL (FT)	FILL CLASSIFICATION (USC)	BASE OF NATIVE SOIL/TOP OF ROCK (FT)	NATIVE SOIL CLASSIFICATION (USC)	NATIVE SOIL DESCRIPTION ****	GROUNDWATER DEPTH (FT)
WB-2	Mud Rotary	13	1	ML/SM	16	SC	Qal	***
WB-3	Mud Rotary	40	36.5	ML/SM	48	CL	Qbm/Qal	***
WB-4	Mud Rotary	25	24	ML/SM	78	ML/MH	Qbm	***
WB-5	Mud Rotary	21.3	6	ML/SM	6	--	--	***
WB-6	Mud Rotary	14	5	ML/SM	5	--	--	***
AP-11	Air Percussion	25	25	GW	25	---	---	**
AP-12	Air Percussion	29	4	GW	4	---	---	**
AP-13	Air Percussion	39	24	GW	61	SP/GW	Qs	30
AP14	Air Percussion	27	18	GW	62	CL?*	Qbm?*	18
AP-15	Air Percussion	37	32	GW	60	CL?*	Qbm?*	**
AP-16	Air Percussion	40		GW	50	*		36
AP-17	Air Percussion	37		GW	46	*		**
AP-18	Air Percussion	35		GW	42	*		**
AP-19	Air Percussion	32	16	GW	16	---		8
AP-20	Air Percussion	35		GW	>50	*		**

*No cuttings returned
16.5* "Depth to Drilling Mud" Measurement
**Groundwater not encountered
***Groundwater not measured due to drilling method
**** Qc-Colluvium; Qs-Beach Sand; Qbm- Bay Mud
--- Native soils not present

Existing Filled Areas

Quarry operations have required the removal of a significant amount of overburden since mining began at the site. Some has been sold as fill, but most has been used to re-contour numerous locations around the property, especially along the current east and south shorelines of the Southeast and Southwest Quadrants. Some quarry overburden is also stockpiled north of the pit on the partially-excavated North Ridge area. Most exposures appear to consist of medium to coarse-grained angular pebble to boulder-sized gravel with a silty to clayey sand matrix, and information derived from the boring logs is consistent with this interpretation.

Available subsurface data indicates that existing fills between the south quarry brow and the Bay margin range in thickness from a few feet up to approximately 60 feet thick. Available blow count data indicates that the fills range from loose to very dense in consistency. Thickness contours of the existing fills, estimated from available boring logs and from air-percussion borings drilled for this study, are presented on Figure 5.

However, since most of the existing quarry spoil deposits were not rigorously compacted or engineered at the time of placement, it is recommended that more precise geotechnical studies of existing fill areas be carried out on a site-specific basis when the Development Plan for the post-mining Second Uses of the site is prepared.

Surficial Soils, Colluvium

It appears that, prior to mining, upland areas on property were mantled with thin deposits of surficial soils developed from weathering of the bedrock and thicker deposits of transported colluvial soils in swales and near the bases of slopes. Most of the areas around the quarry pit have been so extensively modified, however, that these native soils have either been removed or buried by quarry spoils. Relatively undisturbed peripheral upland areas of the property around the South Hill and at the northeast portion of the site do appear to retain native soil deposits, but they were not studied in detail for this study since the intention is to leave them in place for the duration of the mining process. Stiff sandy clays interpreted to be colluvium were noted below fills and above bedrock in Borings WB-2 and WB-3 at the south brow of the quarry pit. Deposits of surficial soils and colluvium can be removed or otherwise mitigated during grading for site reclamation and their presence should not affect future development.

Bay Margin Soils

Some of the overburden generated by the quarry since the beginning of operations has been deposited on areas south of the quarry pit that were formerly inter-tidal or below msl. Most are now roughly at 10 to 20 feet above msl. Around much of the east and south shorelines, the fills are underlain by Bay margin soils consisting of compressible silty clay ("Bay Mud") with local lenses of sand. The thickness of Bay margin soils varies from zero to over 50 feet along the south shore and up to as much as 90 feet along the east shore. Sandy native soils interpreted to be possible former beach or shoreface deposits were encountered in several borings south of the pit as summarized above.

Weathering Profile

The bedrock areas on the property that have not yet been subjected to mining are typically mantled by a weathering profile consisting of a few feet to several feet of silty to sandy clay underlain with 30 to 60 feet of highly to moderately weathered rock.. The weathered rock is typically brown to orange brown. Near the surface, the weathered rock is closely fractured with seams of clay. The degree of weathering decreases with depth, varying with the topography and rock characteristics. The weathering process has typically altered the rock framework minerals, producing clays and oxides which degrade rock quality. The unweathered rock is typically gray to olive gray as described below. The weathered portion of the rock mass has been completely removed over the existing pit area and partially or completely removed in much of the area north of the pit. At the south pit brow, the weathered portion of the rock is still present along the central portion of the quarry pit as shown on Figure 9.

Franciscan Complex

According to Blake, et al. (2000), the rocks at the SRRQ are Cretaceous-age metasediments of the Novato Quarry Terrane of the Franciscan complex. The rocks of the Franciscan complex were tectonically emplaced within the Coast Ranges concurrent with subduction at the edge of the North American Plate. The deformation history of the Franciscan complex rocks has extended from the Cretaceous through several different episodes of faulting and folding. As a result, Franciscan rocks are structurally complex; according to Blake (2000), the mappable Franciscan complex terranes are both fault-bounded and internally folded and faulted. The exposures of Franciscan rocks at the SRRQ are relatively intact by comparison to other terranes within the Franciscan complex. As described below, stratigraphic units of metagraywacke and metashale, although offset by faults, are identifiable and traceable within the pit.

Meta-Graywacke

The mineable deposit at the San Rafael Rock Quarry consists of dense, gray to olive-gray feldspathic meta-graywacke sandstone. According to Blake, et al. (2000), the meta-graywacke is arkosic, containing a significant fraction of potassium feldspar and dark-colored rock fragments, and was derived from a granitic or rhyolitic source. Petrographic and X-ray diffraction analyses of the meta-graywacke performed for rock quality evaluations have indicated that the fines fraction of the sandstone is dominantly white mica and chlorite, with a minor clay component. It appears that the cement between quartz grains consists predominantly of fine-grained non-calcareous minerals. As described above, rocks in the quarry area have been metamorphosed to prehnite-pumpellyite grade (Blake, et al. 2000).

The meta-graywacke is cut by calcite veins ranging from hairline thickness to a few feet thick. Vein orientation is variable and generally sub-parallel to rock joint sets. Most joints are coated or filled with fine-grained white calcite. The most prominent calcite veins are associated with

northeast-striking, northwest-dipping faults that offset bedding up to a few hundred feet (described in more detail below).

The meta-graywacke is generally massive to indistinctly-bedded, with bedding locally defined by thin meta-shale inter-beds. Bedding surfaces are typically undulating and rough. The bedding typically strikes northwest and dips steeply to the northeast. It appears that the former trend of San Pedro Hill was approximately parallel to the strike of the formation and was the surface expression of the massive mineable meta-graywacke sandstone unit. Based on the exposures available at the site and on existing core data, it appears that the mineable meta-graywacke extends to a depth of at least 550 feet below sea level.

Meta-Shale and Meta-Siltstone

At the north pit brow, the meta-graywacke is overlain by a thick sequence of meta-shale and siltstone. The meta-shale is typically dark gray to black, thin-bedded and highly fractured. The frequency of interbedded black meta-shale beds increases from thin and widely-spaced in approximately the upper one-third to one-quarter of the north quarry wall to thick and closely spaced at the brow, where the formation is almost entirely composed of black meta-shale. Further north, scattered exposures consist of a mixture of meta-shale, dark greenish-gray meta-siltstone and meta-graywacke. The trend of the thick black meta-shale unit appears to parallel the trend of a saddle that formerly bounded the north flank of San Pedro Hill. Based on exposures and the former saddle location, it appears that the meta-shale unit is approximately 300 feet thick, although the apparent thickness has been increased by faulting, as described below.

Discussion of South Quarry Brow Subsurface Soil Conditions

The WCC report of 1981 made reference to the close proximity of the south quarry brow to Bay margin soils buried below quarry spoil fills. The area discussed by WCC is adjacent to the area of weathered rock visible on the south quarry brow, as shown on Figure 9. Subsurface

conditions in this area are depicted on Figures 10 and 11. The mapped extent of quarry fill in this area is depicted on Figure 5.

The 2004 ENGEO report identified a filled embayment near the south brow of the quarry bowl using historic maps, previous borings and five additional air percussion borings drilled in a supplemental study in response to comments by OMR. in 2004. For this supplemental study, five additional air-percussion borings were advanced in the area of the northernmost extent of the filled embayment. The contours depicting the elevation of bedrock were adjusted slightly based on the additional borings, as depicted on Figure 5.

Based on the borings, it appears that the bedrock surface slopes southward away from the quarry at a general inclination of approximately 5:1 (horizontal:vertical). The bedrock is overlain by Bay Margin soils as described above and by fill. The Bay margin soils are thickest at the south margin of the site and decrease in thickness to the north. Topography shown on maps from 1895 suggests that the center of the embayment was probably a sandy beach. This was confirmed by borings taken in the central portion of the embayment that encountered natural-appearing sands and rounded gravel. The subsurface information suggests that the sands and gravel deposit is lense-shaped in profile and become thinner to the south. The deposits identified in borings as Bay Mud appear to overlie the sands and gravels in areas away from the former beach area. Above the shore, the 1895 topography indicates that the ground rose rapidly into a swale area. Borings in this area have encountered stiff clays and clay-sand gravel mixtures described as colluvium and alluvium. The original, natural soil profile has been extensively modified by the historical activities of the quarry. These activities have included cuts that partially or completely removed native soils, in some areas extending well below sea level, and fills that have covered native soils. We note that, in quarry investigations, it is quite common to encounter areas that were initially cut and then filled at a later time. This appears to have occurred extensively at the south brow. Our interpretation of the current soil profiles, based on available information is depicted on the Cross Sections, Figures 10 and 11.

In the grading proposed in ARP 04, the finished elevation of the near level area south of the brow of the quarry and north of the shoreline is shown at elevations from +10 to +30 feet msl. This area will be re-configured by making both cuts and minor fills. The proposed cuts are deep enough at some locations to strip away existing quarry fills to bedrock. Where the depth of existing fills exceeds the depth of proposed cuts, or where fills above existing grade are proposed, the new surface will consist of fill.

Review of previous borings logged by previous consultants and ENGEO and the ten new borings recently drilled south of the quarry brow indicate that the excavations proposed in ARP 04 at the south brow and face will expose unweathered greywacke for most of the face area, with a relatively small area of weathered rock just below the brow near the middle of the face in the same areas where weathered rock has been depicted on Figure 9. As described above, the central part of the area, south of the quarry south wall, is underlain by 20 to 60 feet of fill and remnants of colluvium deposits from the pre-mining slopes. Based on the available boring data, it appears that at the projected finished grades a small segment of the pit wall above the proposed ramp will expose the soils described above. This exposure will be local and will be limited to a width of approximately 450 feet and will extend from a brow elevation of +10 msl to a lower interface at approximately -15 msl. This relationship is depicted on Figure 13. Preliminary recommendations for mitigation of soil conditions under this portion of the pit brow are presented in the Recommendations section of this report.

The soil conditions discussed above are a consideration for a small portion of the uppermost part of the quarry face above the proposed access ramp and can be mitigated as part of the preparatory grading that will occur when mining is extended to the south face. The rock mass conditions in the south brow are described in detail below.

Groundwater Conditions

The level of the groundwater in the immediate vicinity of the existing pit walls has been drawn down sharply to below the elevation of the toe of the pit slope. In the summer, standing water fills only the existing pond at the east end of the pit. The quarry operator reports that the water level in the pond and rises slightly in the winter. According to quarry personnel, the water in the quarry pond is used for dust control, and is drawn down significantly in the summer.

Based on the available information, it appears that groundwater in the areas surrounding the pit is contained in two very different aquifer systems. The unconsolidated surficial deposits, including fill, Bay Margin soils and colluvium are essentially a horizontally-layered system where groundwater fills pores between soil particles. The graywacke bedrock is tightly-cemented with little porosity between mineral grains; therefore any groundwater flow must occur through bedrock fractures. Groundwater has been detected at shallow elevations in the quarry brow soils. However, at the time of our mapping in May 2004 and February to March 2005, little or no seepage was visible from soil deposits at the south brow or from fractures in the quarry walls, with the exception of minor seepage noted at the west end of the quarry, below the reservoir at the quarry brow. The general lack of visible seepage could possibly be attributed to the following scenarios.

- The thick and relatively impermeable Bay Mud deposits mantling the Bay floor around the shoreline areas could be acting as an aquitard, impeding the flow of seawater from the Bay into the bedrock around the quarry margins. If the Bay Mud is preventing any significant transmission of Bay water, recharge would be limited mainly to fresh water flows from surrounding on-shore areas. If this is the case, the gradual lowering of the quarry floor and the associated pumping from the pond in the floor could have significantly dewatered the surrounding rock mass, and the groundwater observed in the surrounding soil may be perched in localized aquifers.

- The fracture systems in the rock mass away from the quarry walls are essentially tight and "healed" by the calcite fillings noted in quarry exposures. In this case, groundwater could "head-up" steeply away from the quarry walls. In our slope stability models, we have adopted

the conservative scenario and assumed that the piezometric surface in the surrounding rock mass rises steeply from the quarry floor elevation to approximately sea level.

The existing information on the groundwater level around the existing quarry brow is limited due to the drilling method ("wet" rotary wash methods) used in most of the geotechnical borings from previous studies. The few auger probes performed either did not encounter groundwater or met refusal in rock at shallow depths. Previous studies did not include instrumentation to detect or monitor groundwater levels. Free groundwater was noted in only one of the air-percussion borings drilled for this study on the north brow, Boring AP-8 at a depth of 74 feet. In the ten air-percussion borings recently drilled at the south brow area, saturated conditions were encountered in some of the borings in the granular fill deposits near the elevation of sea level, while in others groundwater was not encountered or could not be measured due to hole conditions. The depth of free groundwater encountered in air-percussion borings is noted above and in Appendix C. According to observations made by the quarry operator, groundwater has typically not been encountered during routine drilling for blasting purposes to depths of 60 to 90 feet below existing benches. For the purpose of analysis of the existing and proposed mining conditions, we have depicted the groundwater as a parabolic surface that varies from slightly above sea level at the brow to just below the surface of the (existing and proposed) quarry floor. The elevation of groundwater assumed for slope stability analysis is depicted on Figures 10 and 11 and on the slope stability output included in Appendix B. The high groundwater levels assumed for our analyses result in significantly lower calculated factors of safety than would be calculated using low groundwater levels. Based on the observed site conditions, we feel that this is an appropriately conservative assumption.

Comments provided by OMR expressed concerns regarding the possibility of a "hydrologic connection to the Bay." In fact, for slope stability analyses we have assumed that there is a continuous ground water surface extending from the Bay through the quarry brow, which is essentially a "hydrologic connection". However, it is important to point out that the groundwater exists within the pores of soil deposits and fractures in the bedrock, and for Bay waters to travel

from the shoreline to the pit, they must pass very slowly though these pores and fractures for horizontal distances of between approximately 450 to 1,000 feet. As a result, the hydrologic connection between the Bay and the pit has never produced significant groundwater flows. The proposed changes to the pit will not significantly change the distance from the pit to the Bay, and therefore, it is unlikely that the overall flow of groundwater into the pit will be greatly increased by the proposed mining plan changes.

As described above and depicted on Figures 5, 10 and 11, unconsolidated soils consisting of quarry fill, colluvium and alluvium appear to extend from the Bay shore through the existing south brow. The hydraulic conductivity of such soil deposits is typically orders of magnitude greater than the typical conductivity of a fractured rock system of the type at the quarry. Therefore, the greatest potential subsurface flows would be expected to occur through the soil deposits rather than the rock. It is significant to note that there is currently no significant seepage in the slopes in the area closest to the old embayment, even though unconsolidated materials apparently extend to the existing south face at elevations below sea level. This observation would suggest that the flow rate of groundwater through the soils at the brow area is currently not very high, and that the hydraulic conductivity of the soils there is relatively low. So, although an existing hydrologic connection between the pit and the brow area is implied for study purposes by our geologic interpretation and in the groundwater surface used for our analyses, the lateral movement of water through the connection is not currently an impediment to quarry operations.

The maximum flows that could occur through the soil layers are limited by the thickness and lateral extent of the soils at the brow, as well as by the hydraulic conductivity of the soil. Based on Cross Sections 3 and 4 and on the estimated lateral extent of soil deposits as depicted on Figure 13, a conservative estimation of possible flows through the soils deposits can be calculated as follows:

The surface area (A) of the potentially saturated soil layer that is projected to be exposed in the face is estimated at between 5,000 and 10,000 square feet (roughly an area 500 feet wide

by 10 to 20 feet high). The gradient behind the face (i) was assumed to be about ten feet in one hundred feet or 0.1. The hydraulic conductivity (k) of the soil was assumed to be 10^{-3} cm/s, which is equivalent to clean, coarse sand. According to Darcy's Law the anticipated discharge would be:

$$\text{Discharge} = k\ i\ A$$

With the above assumptions, the calculated flows are between approximately 7 to 14 gallons per minute.

Flows of this magnitude or even an order of magnitude higher could easily be carried from the quarry brow to the pit by a subdrain system as recommended below. We note that it is highly unlikely that continuous flows at the rate calculated above could be maintained by the soil aquifers for a long period of time. The Bay margin will be approximately 850 feet from the proposed quarry brow and the recharge would occur through soil layers that consist of both clay and silty granular soils. Under constant drainage, the portions of the aquifers near the brow would be drawn down and partially dewatered.

Based on the above calculation, it does not appear that lateral groundwater flow will be a significant impact to reclamation activities if ARP 04 is implemented. Recommendations are given below for treatment of soils exposed in proposed pit slopes. In the event that the rate of flow of groundwater into the pit is increased as a result of new excavations, the recommendations provide means to intercept and redirect the flows in a controlled manner. Any flows directed into the pit could be used for operations. After pit flooding, groundwater flow will not be a consideration.

Effects of Pit Flooding

It is anticipated that the quarry pit will eventually be filled with water after mining is complete, regardless of post-mining site use. ARP 04 calls for flooding to be achieved by construction of a

waterway connecting the pit to the Bay. Even if the property were never reclaimed after mining, we anticipate that flooding would occur naturally over a period of years by the accumulation of rainwater and natural processes including surface runoff.

As described above, for the slope stability analyses in this report, we have assumed that groundwater is present at or above sea level around the existing and proposed pit and very close behind the existing and proposed pit walls, drawn down by pumping. In the current, drawn-down state, the water in the walls of the quarry must conform to a steeply curved surface or "drawdown curve" as described above. Since water is a fluid, it exerts pressure in all directions on the walls of pores and fractures that is equal to the weight of water (62.4 pounds per cubic foot) times the height of the water column (this is close to the same as the wall height). In the sloping walls of the open pit, this pressure is resisted by the weight of surrounding groundwater and rock in all directions except the side of the wall that faces the open air. So, when the pit is open and un-flooded, there is an un-equal pressure acting to push the walls of the pit into the open hole that must be resisted solely by the weight and shear strength of the rock walls. This un-equal pore pressure is considered when the stability of the pit walls in the open condition is calculated. It is for this reason that the computed factor of safety is at the lowest point during mining. It should be noted that this condition will only exist for a short period of time, near the conclusion of mining activities when the quarry property will still be privately owned and maintained. Filling of the quarry with water will essentially begin as soon as mining activity stops and the artificially-maintained low groundwater level in and around the pit is allowed rise. After flooding, groundwater will completely fill pores and fractures in the rock, eliminating the steeply drawn-down depression in the surface of the groundwater that is currently maintained by pumping in the open pit. After flooding, the un-equal pore pressure will be eliminated, since the pressure of the water in the flooded pit will offset and equalize the pressure of the water in the pores and fractures in the walls. In the flooded condition another effect that must be considered is a reduction in the frictional strength along fractures in the pit walls caused by buoyancy. We account for buoyancy in fully saturated (flooded pit) cases by using the buoyant weight of the

rock to calculate frictional strength along joints. After flooding, the elimination of the un-equal inward-acting pore water pressure will greatly increase the factor of safety in the pit walls. These effects are reflected in the calculated factors of safety presented below.

DISCUSSIONS WITH QUARRY MANAGERS

As part of our background research on past quarry slope performance, had discussions with both the existing quarry manager and most recent previous quarry manager. For this supplemental study, we contacted the previous Quarry Manager, Mr. Bob Woodbury, and discussed his experiences, especially in relation to the wedge failure on the south wall, designated W-1 in the discussions below. Prior to contacting Mr. Woodbury, we obtained the quarry topography from 1998 and from 1999. Based on the topography, it is apparent that the wedge failure occurred in the interval between the overflights used for the generation of the map contours. Mr. Woodbury confirmed that the failure occurred during that period, during the summer months of 1998. He did not recall the exact date, and no specific records exist to further narrow the time period during which the failure occurred.

Mr. Woodbury was quite certain that the failure did not occur during or anytime close to a period of rainfall. His recollection was that the wedge movement was triggered when the quarry was in the process of extending a 90-foot bench system from the east to the lower portion of the south face. At that time, the overflight topography shows that the quarry floor was at an elevation of approximately -140 msl. He stated that the drilling and blasting was in process on a bench at an elevation of approximately –100 msl near the location of the wedge. The blasting was extended to a depth of approximately -140 msl. The likely bench and face configurations at that time, reconstructed from the 1998 and 1999 overflights, is depicted on Figures 11 and 12. Mr. Woodbury was present at the time that the wedge failed, and he recalled that a blast had just been completed and a worker was in the process of "mucking" the loose, blasted material directly at the toe of the wedge.

Mr. Woodbury stated that the failure was triggered immediately after the worker removed some blasted rock from the toe of the wedge, and that the movement was relatively rapid. Based on Figures 11 and 12, it appears that the failure extended from the toe, through two bench faces, and

into a flatter area at elevations of -50 to -25 msl, as well as a portion of an old face extending from approximately-25 msl to approximately 10 msl. No movement was noted at the quarry brow at approximately 25 msl. The failure deposited a cone of debris on the benches at -100 msl and -140 msl. A portion of the debris cone is still in place, covering portions of the benches at -100 msl and -140 msl.

According to Mr. Woodbury, the failed wedge consisted of numerous blocks of broken rock. In order to improve the stability of the failed material, he set up a dragline at the quarry rim to remove some of the failed material and to create the relatively smooth surface that is visible today.

Mr. Woodbury stated that no groundwater seepage was noted in the wedge area during blasting or subsequent stabilization activity. Mr. Woodbury could recall no other incidents of slope failure during his 11 years as quarry manager. Since the movement of the wedge, mining has been limited to the lower benches on the south face, below -140 msl. No movement has been noted since the removal of the upper wedge material, according to Mr. Peer.

Current Blasting Practices

The current owner, Dutra Materials, has operated the quarry since 1986. In that time, all four faces have been mined to differing extents. Over the last 4 years, the bench configuration in areas of active mining has been changed from 45-foot-high faces with 30-foot-wide benches to 90-foot-high faces with 30-foot-wide benches. The current Quarry Manager, Mr. Brian Peer, has instituted a program to improve blasting practices concurrent with the change from 45-foot-high faces to 90-foot-high faces, since he took over management in 2000. This program has been carried out extensively on the east, north and west quarry faces since 2000.

Mr. Peer observed that previous blasting practices tended to create excessive overbreak and commonly resulted in weakened faces with overly steep or overhung inclinations. He has upgraded blasting practices to include the following:

- Prior to creating a new bench face, the proposed alignment is "pre-split" by drilling a closely spaced line of blast holes at the proposed new bench alignment. The alignment is blasted with charges calculated to create a fracture line at the location of the desired face with minimal disturbance of the proposed permanent face.

- Prior to blasting, spoils are removed from the toe of the slope to be blasted, to provide a free and un-obstructed space for displacement of the blasted material. This practice minimizes overbreak of the material behind the blast charges.

- Blasting at the working faces is staged in increments that limit the width of the blasted zone. This practice is designed to remove material in a manner that minimizes overbreak.

Since institution of improved practices, Mr. Peer has noted better control of face orientation and minimal overbreak and degradation of face integrity. During our quarry reconnaissance in 2004, we noted that the actively-mined faces tended to have uniform bench heights and relatively uniform and intact-appearing faces. In contrast, the mid to upper benches at the south face, un-mined since approximately 1998, exhibit many overhung and highly fractured faces with varying face heights and bench widths. The highly fractured and degraded rock in these faces will be excavated and reshaped using improved mining practices as the process of deepening the quarry to -350'MSl is implemented. During his time as the Quarry Manager, Mr. Peer reports that he has not experienced any significant bench failures or slope stability problems. He attributes the better performance at least partially to the improved blasting practices.

ESTIMATION OF SEISMIC PARAMETERS

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site. The degree of shaking is dependent on the magnitude of the event, the distance to its zone of rupture and local geologic conditions. For geotechnical analysis and design purposes, the peak ground acceleration (PGA) can be estimated by either "deterministic" or "probabilistic" methods. For this study, the PGA was estimated probabilistically.

A probabilistic estimation of PGA considers all of the known seismic sources in the site region and the estimated activity rate of each source to calculate a statistically estimated PGA based on a probability of occurrence. We performed a probabilistic seismic hazard evaluation for the site. In this analysis, a computer program (EZ-FRISK) was used to model the seismic setting of the region and is able to explicitly account for uncertainty relating to:

- Earthquake magnitude.
- Rupture length.
- Location of rupture.
- Published maximum earthquake magnitude.
- Published recurrence interval of earthquake events.

The program calculates, by summation from earthquake sources, the total average annual expected number of occurrences of acceleration greater than each of several specified values. Once the annual probability is obtained, the probability of the level of ground acceleration being exceeded over a specified time period is calculated. Using this method, a horizontal ground surface acceleration of 0.60g is predicted to have a 10 percent probability of being exceeded in a 50-year design life. The probabilistic ground surface acceleration was derived using current published seismic source information (WGEP, 2003) and attenuation relationships developed by

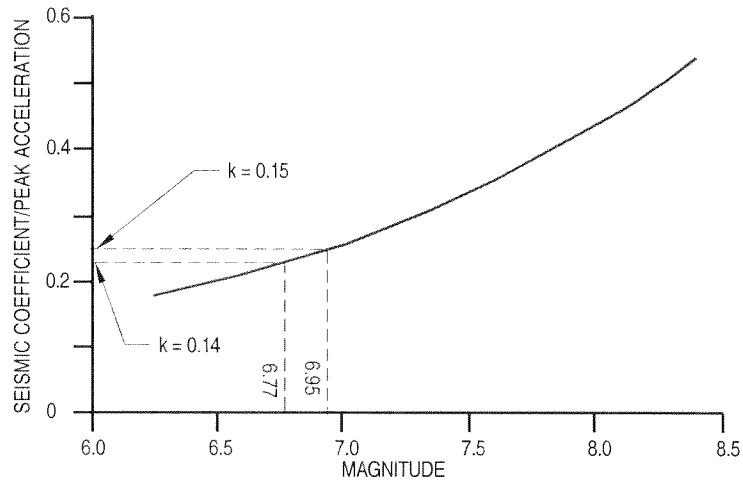
Boore, Joyner and Fumal (1997), Abrahamson-Silva (1997), Atkinson-Silva (2000) and Sadigh (1997).

Selection of Seismic Coefficients

Seismic coefficients for pseudo-static slope stability analyses were selected based on the type of analysis and on site-specific conditions including the PGA and slope configurations. Based on the site conditions, separate seismic coefficients were for analyses of slope face stability and for analyses of the global slope stability as described below.

- The effects of topographic amplification on the stability of slope faces were evaluated according to the method of Ashford and Sitar (2002). This method was developed based on observations of shallow slope failures in coastal bluffs following the 1989 Loma Prieta Earthquake. According to the authors, this method is applicable for shallow "thin-skinned" brittle failures in un-consolidated soils (such as coastal terrace deposits) in steep slope faces. The method considers slope height, PGA and the ratio of the expected failure to the slope height. The assumptions used in our analysis are included in Appendix B. Based on this method, we calculate a seismic coefficient of 0.19. We note that, based on the description of the assumptions that Ashford and Sitar used in formulation of this method, it does not appear to be well suited to evaluating seismic amplification in dense, hard rock such as the SRRQ greywacke. However, as requested by OMR, the calculated seismic coefficient was applied to analysis of wedges in SWEDGE© as described below. Supporting calculations are included in Appendix B.

- The seismic coefficient for deep circular analyses was estimated based on Pyke (2004). This method calculates a seismic coefficient from PGA for a given earthquake magnitiude based on the curve illustrated below. According to Pyke (2004), the curve is configured to compute seismic coefficients that correlate to newmark displacements in the "acceptably small" range, based on Makdisi and Seed (1978). Based on de-aggregation of the probabilistic earthquake analysis described above, the mean earthquake magnitude for the site is approximately 6.77. Based on the curve, and using a PGA of 0.60, the estimated seismic coefficient for an M 6.77 earthquake would be 0.14. In our experience, the standard of practice would be to use a seismic coefficient of 0.15 for sites in close proximity to Class A seismic sources such as the Hayward and Rodgers Creek Faults. Based on the method described by Pyke (2004), we conclude that a seismic coefficient of 0.15 is conservative and appropriate for analysis of global slope stability at the SRRQ site.


SEISMIC COEFFICIENT/PEAK ACCELERATION
0.6
0.4
0.2
0
k = 0.15
k = 0.14
6.77
6.95
6.0
6.5
7.0
7.5
8.0
8.5
MAGNITUDE

ROCK MASS DATA ACQUISITION AND KINEMATIC ANALYSIS

Field Evaluation

Characterization of the rock mass in the field was carried out by ENGEO Geologists registered in the State of California. Observations and measurements were made in the field from accessible benches and from the quarry floor. Inaccessible areas were characterized by remote observation from appropriate vantage points in the quarry. The data set includes 300 measurements. Field measurements included:

- Measurement of strike and dip of bedding and joint sets using a Brunton© compass at 49 stations around the existing quarry walls. NOTE: in the body of this report and in the attached tables in Appendix strike and dips are reported in the right hand rule convention (RHR). In the attached SWEDGE© analyses, strikes and dips are reported in the dip and dip direction convention (as required by the SWEDGE© data input convention).
- Visual characterization of the surface roughness according to Barton's Joint Roughness Coefficient (JRC) as described in Watts, et al. (2003) and Hoek and Bray (1981).
- Visual characterization of joint surface coatings and infill.

Field observations also included characterization of joint continuity by visual mapping onto the topographic base map, and mapping of prominent faults and bedding contacts on the base map and on panoramic (un-scaled) composite photographs. Field mapping observations and selected orientation measurements are presented on Figures 6 through 9.

Aerial Photo and Map Characterization

Field observations were supplemented with measurements of prominent visible joint sets from aerial photographs (flown in April 2004) scaled and superimposed on the site topographic map, and measurement of large, prominent faces directly from the topographic map. A supplemental analysis

of the bounding discontinuities at Wedge W-1 was included in this report for OMR review. This analysis was based on detailed topography of the wedge photogrammetrically compiled from January 2005 aerial photographs and supplemental geologic mapping. The supplemental topographic map and geologic information are depicted on Figure 12.

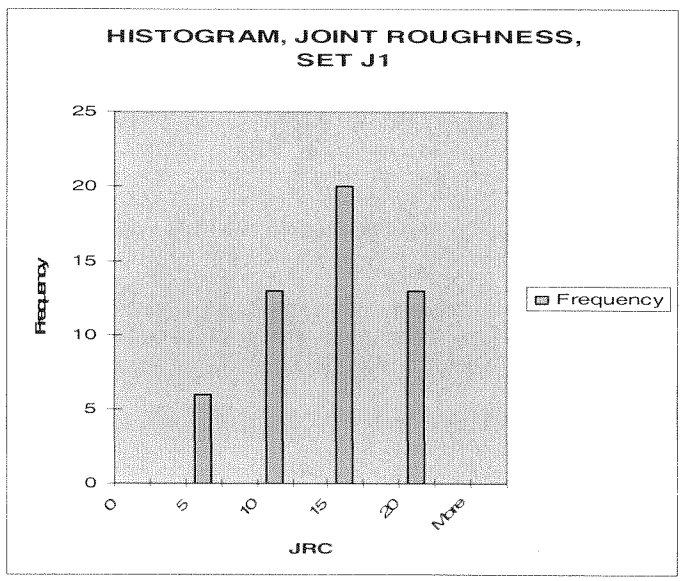
Discontinuity Data Analyses

Discontinuity orientation data was summarized by plotting the poles of measured planar data on an equal-area stereonet using the Stereostat© program from Rockworks, Inc. Data were organized into four data groups based on quarry face orientation.

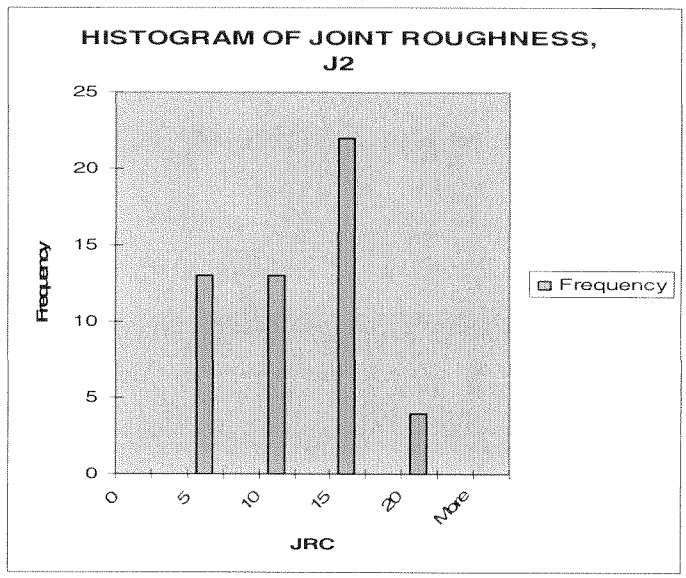
The data plots were contoured using the Stereostat© program to aid in definition of joint sets by attitude. The contoured stereonet plots are depicted on Figures 6 through 9. The contoured plots were examined and the data was compiled based on the grouping of poles to define sets for analyses. The compiled sets were derived by visually picking bounding planes encompassing the range of strikes and dips for each significant cluster of poles as described by Watts (2003). Based on the stereonet analyses and field observations, it was interpreted that the rock mass discontinuities could be grouped into six sets including:

- Bedding defined across most of the quarry (with the exception of the upper north brow) by thin (less than one inch to less than one foot) beds of black meta-shale. As discussed above, a thick shale unit underlies the upper north brow. This contact (mapped on 2004 contours) is depicted on Figure 5. Bedding surfaces are typically undulating to moderately rough, corresponding to a JRC of 10 to 15. In addition, bedding planes have been offset by at least three joint sets by distances ranging from a few inches to up to approximately 200 feet. Bedding orientation is relatively consistent across the quarry, generally striking at 300 to 330 degrees and dipping 35 to 55 degrees to the northeast. Bedding planes defined by laminations or grain-size variations within meta-graywacke beds were observed to be indistinct.

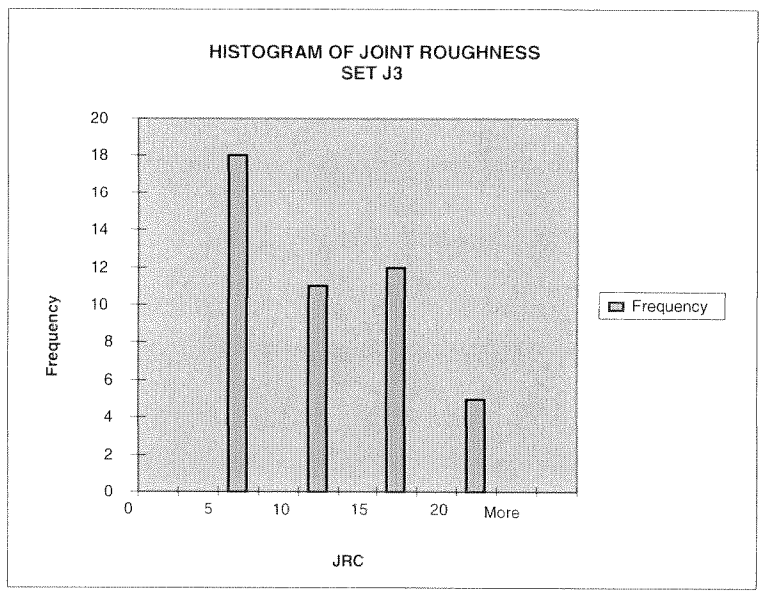
- Set J1 defined as the most prominent joint set generally striking at approximately between 180 and 220 degrees and typically dipping between 50 and 90 degrees northwest. The J1 joint set appears to have accommodated most of the visible fault displacement defined by offset of the base of the thick meta-shale contact at the north quarry brow. This joint set was observed to have numerous members that are continuous for hundreds of feet and a few identifiable planes that are continuous from the north to the south quarry faces. Mappable members of this set are depicted on Figures 5 and 6 through 9. The continuous members of this set with observable offsets are classified as faults, and typically displayed slickensided calcite surfaces. Slickensides were observed to trend and plunge at angles oblique to the strike of the planes. Several planes in this set are filled with sparry calcite veins and calcite-cemented graywacke breccias up to a few feet wide. Joint roughness measurements in this set ranged from a JRC of 5 to 20 averaging approximately 14. The most continuous members had a JRC in the range of 5 to 10. Although the age of faulting could not be definitively determined, it does not appear that the faults are active or very young, based on the extensive calcite mineralization that has essentially "healed" the fracture spaces on most observed joints.



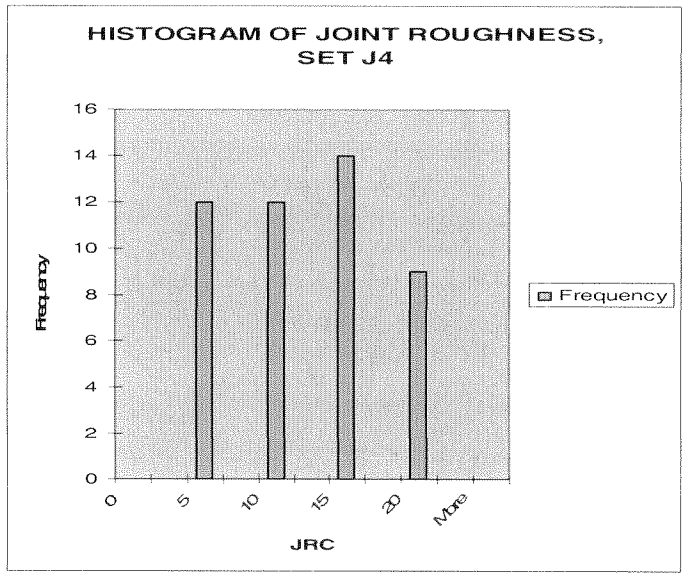
- Set J2 is defined as a prominent joint set striking at approximately 110 to 180 degrees and dipping between 40 and 90 southwest. Members of J2 were locally slickensided and filled with calcite-cemented rock breccia, indicating that they have accommodated tectonic offset. However, this joint set cannot be clearly associated with the major offsets described above for Set J1. Several planes in this set were observed to be laterally continuous for tens to hundreds of feet. At the east quarry wall, members of this set form some of the visible inter-bench faces. Mappable members of this set are depicted on Figures 5 and 6 through 9. Measured JRCs ranged from 5 to 20, averaging approximately 12.



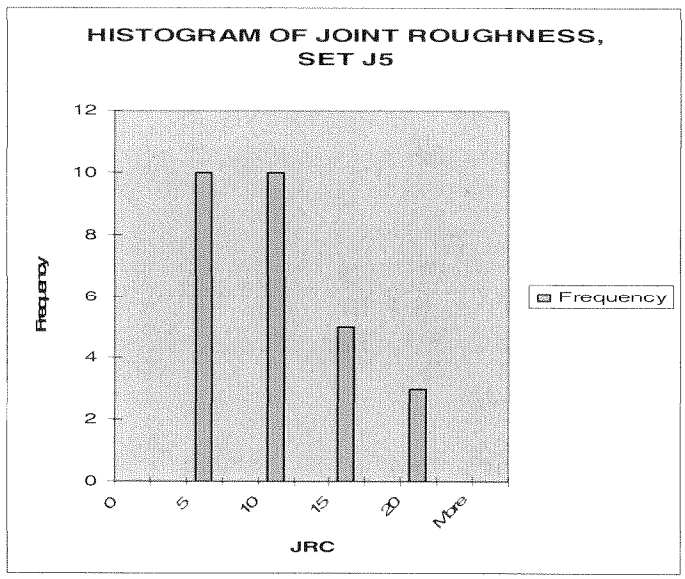
- Set J3 is defined as a prominent joint set striking at approximately between 40 and 80 degrees and dipping between 60 and 90 degrees southeast and 80 and 90 degrees northwest. Many members of this joint set were observed to be horizontally slickensided and filled with fine-grained calcite. Slickensides were observed to be sub-horizontal and well developed. Several members of this set were observed to be laterally continuous for hundreds of feet, and to commonly form steeply south-dipping inter-bench faces on the north quarry wall. The magnitude of offset along the joint surfaces was not easily observable. However, it was noted that laterally continuous members of this set were truncated by other joint sets. Mappable members of this set are depicted on Figures 5 and 6 through 9. Measured JRCs ranged from 5 to 20, averaging approximately 10.



- Set J4 is defined as a moderately prominent joint set striking at between approximately 260 and 300 degrees and dipping at between approximately 50 and 90 degrees northeast. This joint set was observed to form a number of short but prominent faces on the south quarry wall. Mappable members of this set are depicted on Figures 5 and 6 through 9. Measured JRCs ranged from 5 to 20, averaging approximately 12.

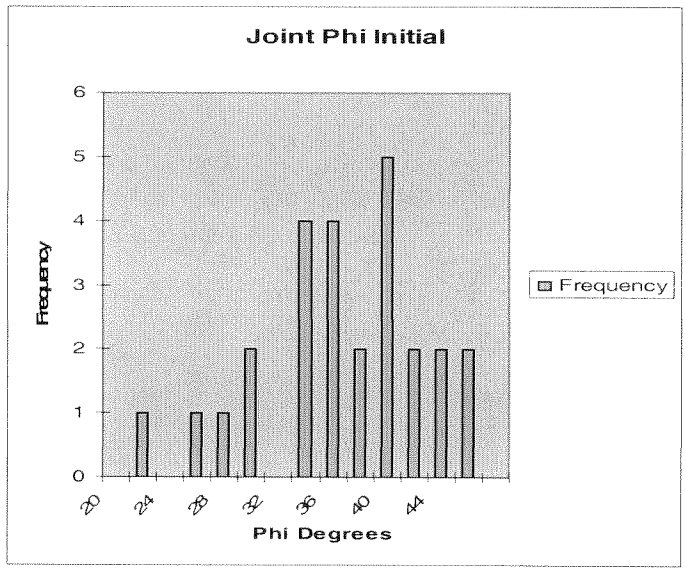


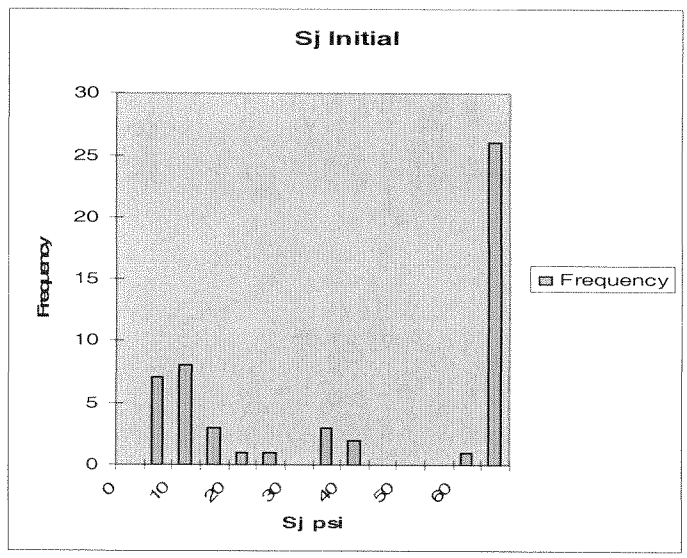
- Set J5 is defined as a moderately prominent joint set striking at between approximately 300 and 360 degrees and dipping at between approximately 35 and 70 degrees northeast. This joint set is nearly parallel to bedding, but appears to be a distinct set, at several locations defined as calcite and calcite-breccia-filled fractures with no included meta-shale. A calcite-breccia filled joint defines one side of the existing wedge failure on south quarry wall, but is distinct in orientation from an adjacent meta-shale filled bedding plane. Mappable members of this set are depicted on Figures 5 and 6 through 9 as orange lines. Measured JRCs ranged from 5 to 20, averaging approximately 10.

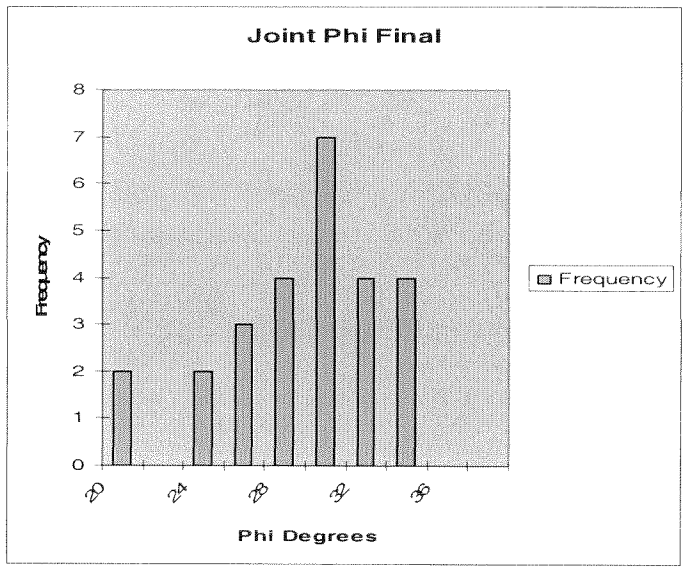


Joint Shear Strength from Yerba Buena Island Testing

In response to OMR comments, we have excerpted data from the Bay Bridge east span replacement study prepared by Fugro-Earth Mechanics, Inc. (FEMI) (2001). The rock sampled for the Bay Bridge study was located at Yerba Buena Island (Figure 3), which is within the Alcatraz Terrane as described by Blake, et al. (2000). The greywacke and shale within the Alcatraz Terrane, like the rocks at the quarry, are described as metasediments of prehnite-pumpellyite grade, with healed calcite fractures Blake, et al., (2000). The FEMI testing program included direct-shear testing of joint samples from cores. Appendix 3C of the FEMI (2001) report is a compilation of rock strength testing by GeoTest, Unlimited (GTU). This appendix is presented as Appendix D of this report. We selected 26 test results from the GTU report that were performed on cores of relatively unweathered greywacke representative of the rock exposed in the pit. The GTU direct shear testing program measured and reported "initial" and "final" joint shear strength expressed a friction angle (phi, in degrees) and a cohesion intercept or Sj (in psi). The initial shear strength is reported by GTU as the strength measured from the original, relatively undisturbed and "mated" joint samples, and is assumed for this analysis to represent the intact, peak joint strength. The final shear strength is reported by GTU as a reduced strength measured on the joint surfaces after shearing and partial breakdown of the joint surface asperities. Only the frictional component of the final joint shear strength is reported below. Histograms of the selected GTU test results are presented below.


Joint Phi Initial
Frequency
0
1
2
3
4
5
6
20
24
28
32
36
40
44
Phi Degrees
Frequency


Sj Initial
Frequency
0
5
10
15
20
25
30
0
10
20
30
40
50
60
Sj psi
Frequency



The averages of shear strengths selected from the GTU testing program are summarized below.

TABLE 2

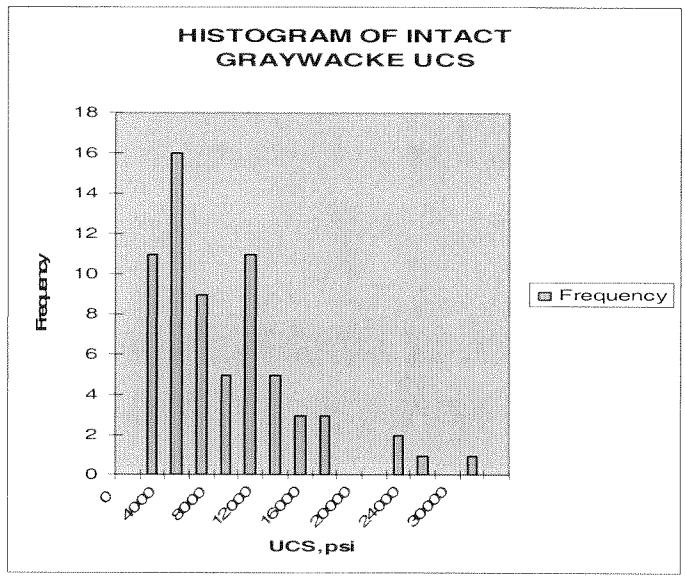
AVERAGE JOINT SHEAR STRENGTH, YERBA BUENA ISLAND		
Joint phi, Initial (degrees)	Joint Cohesion (Sj), Initial	Joint phi, Final(degrees)
37	15 psi (1.08tsf)	29

Intact Rock Strength from Yerba Buena Island Testing

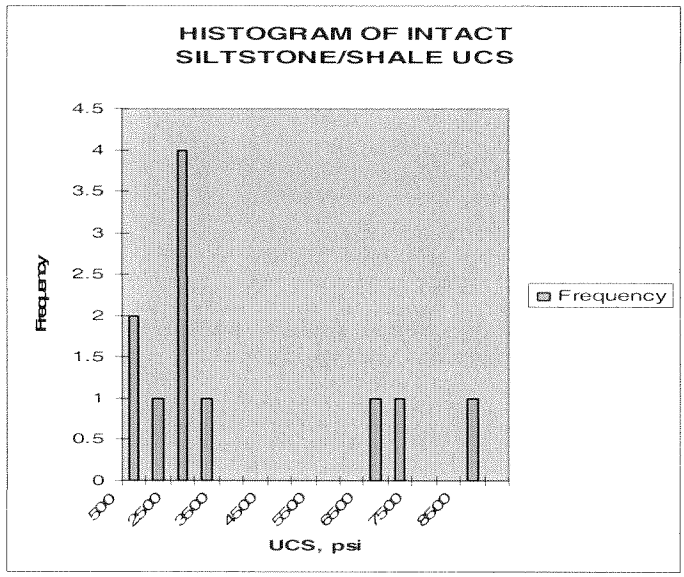
Intact rock strength was estimated based on review of data gathered as part of the Bay Bridge east span replacement study prepared by Fugro-Earth Mechanics, Inc. (FEMI) (2001).

In FEMI, (2001) the unweathered rock is described as gray graywacke sandstone with numerous fine intersecting calcite-filled healed fractures and interbedded dark gray siltstone. GTU performed a total of 111 unconfined compressive strength (UCS) tests on intact rock core samples. We examined the table of unconfined compressive strength test results and selected

67 test results with descriptions applicable to the rock at the quarry. The selected tests were those described as gray, moderately weathered to fresh sandstone with calcite-filled healed fractures. Several selected tests also included thin siltstone interbeds similar to the dark gray shale/siltstone beds found throughout the quarry exposures. UCS values reported by GTU ranged from less than 1,000 psi to 27,000 psi as shown below Based on the distribution of the results of testing, we conservatively assigned a design value for intact rock strength of 7,000 psi.



We also selected 8 UCS tests on relatively unweathered siltstone samples judged to be representative of the shale/siltstone unit at the north quarry brow. The design intact strength for the siltstone was assigned as 3,000 psi.



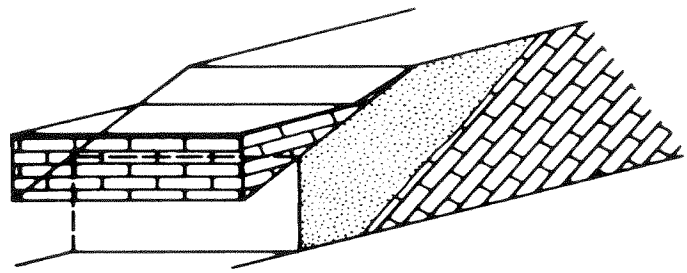
A summary of test results performed for the FEMI study is attached as Appendix D.

Kinematic Analysis of Discontinuities

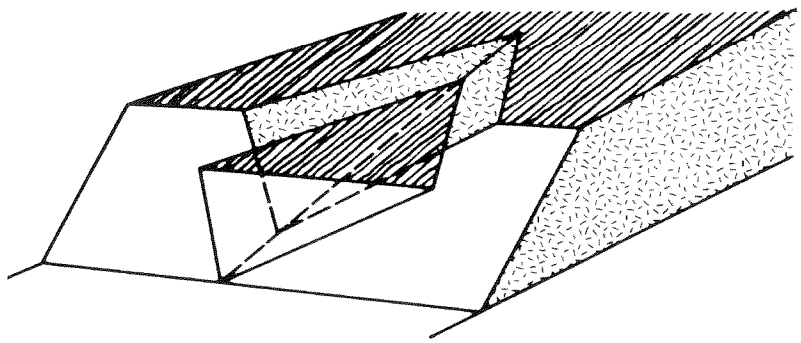
Kinematic joint analyses are essentially an examination of the orientation of rock mass discontinuities with respect to the orientation of existing or proposed rock slope with the intent to detect possible failure modes. Kinematic analyses focus on joint surfaces or combinations of surfaces that project adversely from the existing or proposed rock slope, and therefore, could allow a portion of the rock mass to move into the free space in front of the slope, creating a slope failure. It is important to note that kinematic analyses focus solely on geometry and do not consider the shear strength of discontinuities. The shear strength and continuity of rock joints is a large factor in the stability of any discontinuity-bounded rock mass; a kinematically-permissible failure geometry can in many cases have a high factor of safety against slope movement. Rock slope stability is discussed below.

Common failure modes considered relevant for the current study include plane failures and wedge failures. Plane failures can occur when a discontinuity surface such as a joint or bedding

plane dipping at an inclination flatter than the rock face projects adversely from the slope surface. In general, plane failures occur when the dip direction of the discontinuity falls in a range within approximately 20 degrees (plus or minus) of the slope face dip direction. Outside of the 20 degree zone, the likelihood of plane failure is greatly diminished (Watts, 2003). An example of a plane failure is illustrated in (*a*) below. Another common failure mode is the wedge failure, which can result when two intersecting discontinuities combine to form a wedge-shaped rock mass. If the inclination of the line of intersection, or hinge of the wedge is flatter that the slope inclination, the wedge is potentially free to move out of the slope face, as illustrated in (*b*) below.



(*a*)



(*b*)

Joint orientations compiled as described above, were plotted on the equal-area stereonet along with planes representing the proposed and existing quarry faces, as depicted on Figures 5 and 6 through 9. The joint data are organized into four groups according to the quarry wall where the data was collected. Our field observations and the plotted stereonet data are summarized below:

- East Wall The data from the east wall are summarized on Figure 6. The existing face is dominated by a series of approximately 90-foot-high inter-bench faces. This wall is the most steeply inclined of the existing walls (Figure 10). Quarry production blasting has preferentially split the rock along steeply-inclined, prominent and continuous joints of Set J2, forming prominent exposed faces visible in the photograph in Figure 6. The existing steep and high faces appear to be performing well. Access to this face was limited, but visual examination and collected data suggest there is no apparent well-defined adversely-oriented structure in this face.

- West Wall Joint data from the west wall are summarized on Figure 7. The west wall consists of two faces, oriented as shown on Figure 7. Many continuous steep joints in Sets J2 and J4 are visible in the existing faces. The plotted joint data show a possible daylighting wedge mode formed by Sets J3, J4 and bedding. However, the angle of the wedge hinges is generally flatter than 35 degrees, and no such wedge failures were observed to have occurred. Locally on the northeast-facing portion of the west wall, we noted shale partings/bedding planes forming exposed faces. Bedding-plane failures are a theoretically possible failure mode for this portion of the face based on the measured data. However, as previously noted, at least four of five joint sets, display visible slickensides suggesting that some level of displacement has occurred along the discontinuities. Mappable bedding displacement has been noted on Set J1 and any displacement on other joint sets will have displaced originally continuous bedding planes. The bedding in the west face is poorly defined by widely-spaced, rough-surfaced shale partings. It therefore appears that bedding plane failures may be limited by the displacement along joint planes and by the wide spacing of shale partings. We saw no evidence of large previous plane failures. Above the active quarry brow, there is a steep, natural-appearing east-facing slope that is over 150 feet high. The exposed faces appear to be defined by members of sets J4, J5 and bedding. The slope is stained by weathering and shows no evidence of any extensive recent failure scars.

- North Wall The north wall data are summarized on Figure 8. Exposures on the north wall are dominated by near-vertical inter-bench faces formed from joints of Set J3. Few existing wedges are visible. The most notable wedge structure is formed by the intersection of Sets J2 and J3. A small wedge failure (Wedge W-3) in a steep bench face was noted as pictured on Figure 8. Plotted joint data suggest that the inclination of the wedge hinge will typically be close to or steeper than the proposed 60-degree slope. This geometry would typically not dip adversely out of the proposed slope. Another possible daylighting wedge geometry could combine Sets J3 and J5. The majority of the wedges plotted on Figure 8 would not dip adversely out of the proposed 60-degree slope. In addition, no existing wedge failures combining Sets J3 and J5 were noted. The stability analysis of wedges is discussed below.

- South Wall The data from the south wall are summarized on Figure 9. The rock exposed in the south wall is visibly more weathered near the brow than the other faces, due to the relatively shallow depth of cut from original grade. Visual examination of the south wall reveals numerous daylighting wedges. The most prominent wedges are formed by Sets J1 and J5. The largest existing wedge failure, designated Wedge W-1, was described above. W-1 is bounded by a joint in Set J1 and a calcite-cemented breccia in Set J5. Wedge W-1 and its bounding surfaces are depicted on Figures 9, 11 and 12. Another prominent wedge occurs near the west end of the south wall (Wedge W-2). Wedge W-2 is formed by the same joint sets as Wedge W-1. Only a small portion of the brow of the lowest bench has failed at this location, as shown on Figure 9. A third small wedge failure (Wedge W-4) occurs between Wedges 1 and 2 at the lower bench brow. This wedge is defined by a meta-shale parting on a bedding plane and a joint in Set J1. Numerous other small wedges, generally affecting the outer portions of single benches were noted across the south wall.

We noted no evidence of extensive plane failures. As shown on Figure 5, the dip direction of bedding is generally skewed from the face by 20 to 50 degrees. Plane failures appear to be unlikely except for conditions where irregular inter-bench faces locally create adverse exposures.

The stereonet plots of joint data for the south wall are consistent with the observed conditions. Kinematically-permissible wedge failure modes are defined by the intersection of Joint Sets J1, J2, J4, J5 and bedding planes. The stability analysis of wedges is discussed below.

Rock Mass Characterization

The rock mass characteristics were approximated by estimation of the Geological Strength Index (GSI) as described in Hoek (2000), and by application of the method of Hoek (2000), using Joint Roughness Coefficient (JRC) measurements gathered during our field study.

The GSI was estimated using two methods:

- A high-quality core log from Boring U-5A (Golder, 1991) drilled near the southeast corner of the pit floor was used to estimate the average fracture spacing and Rock Quality Designation (RQD) for the portion of the core between elevations -160 and -450 msl. Based on the method of Bieniawski (1989), we estimated the Rock Mass Rating (RMR) to be in the range

of 45 to 65. Based on Hoek (2000), the GSI is estimated at 40 to 60 (RMR-5). Table 1, below, summarizes the values used for our estimate.

Table 3 Estimation of RMR (Bieniawski, 1989)		
Parameter	Range of Values	Rating
Intact Rock Strength, MPa	20-80	4-7
RQD %	40-60	8-13
Fracture Spacing, mm	150-300	8-10
Condition of Discontinuities	See RMR Chart, Appendix E	10-20
Groundwater	Set at 15 according to Hoek, 2000	15

- We compared the estimated GSI to Table 11.6 of Hoek (2000) which is a method of visually estimating GSI from surface exposures. Based on the table, we estimate a GSI value ranging from 30 to 60, corresponding to a rock mass ranging from "Blocky-Disturbed; folded and/or faulted with angular blocks formed by many intersecting discontinuity sets" to "Very Blocky; interlocked, partially-disturbed rock mass with multifaceted angular blocks formed by four or more discontinuity sets." Table 11.6 is attached in Appendix E.

Based on the kinematic analysis described above, it appears that analysis using the Hoek-Brown failure criterion (Hoek, 2000) is appropriate for limit-equilibrium slope-stability analyses. Based on the above-described methods, we have used a GSI 50 for graywacke in our analyses. A GSI of 25 was used for the shale/siltstone at the north brow, based on Table 11.6 of Hoek (2000).

Assumptions on Blast Disturbance

In the above section of this report describing the observations of past and present quarry operators, we noted that the current quarry manager has implemented a program to improve blasting practices. The effects of improved blasting techniques are already apparent in the actively mined north, west and east faces, which exhibit reduced overbreak and better control of bench face orientation and inclination. The older portions of the south face are visibly more disturbed.

According to Hoek, et al. (2002), blast disturbance can have a significant effect on the strength of rock masses. In order to account for this effect, the Hoek-Brown failure criterion, as incorporated into the program SLIDE© includes a rock disturbance factor D that can range from 0 for undisturbed rock masses to 1 for rock masses such as those in very large open-pit mines that have been subject to heavy excavation disturbance related to poor blasting and overburden stress relief. The guidelines for estimation of D presented in Hoek, et al. (2002) and included in SLIDE© are based on experience and published analyses from a number of mining sites around the world. We note that the quarry, even at the proposed maximum depth of -400 msl is a relatively small excavation when compared to the types of large, open-pit mines that are considered in Hoek, et al. (2002).

In keeping with the level of conservatism that we have applied to this preliminary analysis, we used the maximum disturbance factor (D = 1) in our calculations. We note that in Table 1 of Hoek, et al. (2002) a disturbance factor of D of 0.7 is considered appropriate for smaller scale projects where good blasting practices are employed such as SRRQ. We also note that the disturbance to the rock mass due to blasting and overburden stress relief will diminish with distance from the quarry pit. However, in the preliminary slope stability models presented in this study, we have assumed a uniform level of disturbance for the entire rock mass. In future analyses, as recommended below prior to the final site use, it should be possible to apply a more detailed rock disturbance model based on detailed mapping of finished slope faces and fracture spacing data collected from boreholes. For comparison and review purposes, Section 2 was analyzed using lower disturbance factors, as discussed below.

DISCUSSION OF MINING-CONCURRENT AND POST-MINING SITE USE ASSUMPTIONS

The assumptions about acceptable risk that are inherent to judgments on the appropriate level of stability for the quarry pit under mining conditions are fundamentally different than those that apply to the post-mining site uses. The section that follows outlines the criteria for each.

Mining-Concurrent Risk Acceptance

In general the following assumptions apply to the mining case:

- The mine should not create an undue hazard to the surrounding properties or to the public either during or following mining.

- Mining is a resource extraction operation that occurs in the context of rough, and often temporary or movable improvements such as haul roads, mining faces, and rock handling and processing equipment. The Surface Mining and Reclamation Act (SMARA) does not provide a specific standard for slope stability factors of safety under mining conditions. During mining, mine safety is governed by the Bureau of Mines. In practice, however, quarry operators commonly accept slope stability factors of safety that are close to unity, such as 1.1 for non-critical slopes (no mine structures or permanent haul roads threatened). In many circumstances, mines are operated safely with unstable slopes that are carefully monitored for displacement behavior.

- The assumptions used to estimate slope stability risk during mining can be refined based on performance and on-site experience. If slope performance is judged to be below an acceptable standard, the mining practices can be adjusted to improve performance, and/or mitigation procedures can be implemented.

- The practice of mining carries an inherent economic risk to the mine operator in the case of a relatively uncommon event such as a large nearby seismic shaking event. Due to the economics of mine operation, it is not practical to design mine excavations with high factors of safety under seismic loading conditions. Mining operators customarily factor clean up and down-time losses into their business plans in seismically active areas. Since the SRRQ quarry pit is not located in close proximity to properties or improvements owned or operated by any third party, the consequences of mining failures would impact only the mine owner.

- Wherever possible, the factor of safety of conditions in place at the completion of mining should be appropriate to those required for the intended post-mining land uses of the site.

Post-Mining Risk Acceptance

Where post mining use of the property includes real estate and human occupancy uses the factor of safety of the finished slopes must satisfy a higher standard that is sensitive to relatively small vertical or horizontal displacements. In our experience, the following standards generally apply for the kind of Second Uses anticipated for the SRRQ site:

- Habitable structures and lifeline utilities should be designed to perform in accordance with generally-accepted codes intended to protect improvements under normal circumstances and to protect life safety under rare circumstances, such as large earthquakes.

- Second Uses must meet stringent local and State grading and construction codes and ordinances.

- In the San Rafael area, as well as the Bay Area as a whole, the generally accepted factors of safety for design of permanent slopes in human occupancy projects is 1.5 for static conditions, and 1.1 to 1.15 for pseudo-static conditions. In addition, some form of seismic deformation analysis is sometimes performed to demonstrate that damage to improvements would be minimized in the event of a seismic occurrence.

- The geotechnical design and expected performance of improvements is tied to a "standard of practice for the Bay Area" which is based on the typical practices of the majority of geotechnical consultants.

Back Analysis of Selected Wedges

Four existing wedges (designated 1, 2, 3 and 4) were back-analyzed using the program Swedge© from Rocscience. The locations of the wedges are depicted on Figure 5. For the purpose of back-analyses, we made the following conservative assumptions.

- The shear strength along the back-analyzed joint surfaces was frictional with no cohesion contributed from interlocking sections of intact rock. The assumption of a frictional strength with no cohesion is, in our opinion, appropriate only for small rock masses bounded by locally smooth and continuous joints. The large, quarry-wall scale rock masses that are considered below in the forward-analysis portion of this analysis are bounded by discontinuity sets that encompass much larger surface areas than the small back-analyzed wedges. The shear strength of large-scale discontinuities will, in addition to friction, include cohesion contributed by other factors such as the limited lateral extent of individual joints, undulations on joint surfaces and tectonic offset of joints by intersecting joints. All of these factors will increase the shear strength joints by introducing asperities and "bridges" of intact rock.

- The back-analyzed wedges failed due to the weight of the wedge, i.e. that rock disturbance from blasting or excavation activity did not play any role in the movement of the rock mass above the joint planes. This assumption is conservative, since the rock exposed in the vicinity of the back-analyzed wedges was visibly affected by production blasting, and it is possible that all or portions of some of the smaller back-analyzed wedges actually failed during blasting and/or during subsequent mechanical excavation. Neglecting disturbance in back analysis results in calculated shear strengths that are lower that would be calculated if disturbance were included. It is therefore likely that the back-analyzed shear strengths reflect a disturbed rock condition that is actually weaker than the "global" rock mass conditions. At the request of OMR, a more complex set of assumptions was used for the case of wedge W-1, as described below.

- The back-analyzed wedges were assumed to be dry, i.e. no pore water pressure was included in the analysis. Neglecting pore-water pressures in back analysis results in calculated shear strengths that are lower that would be calculated with pore water pressure, although it is possible that at least one back-analyzed wedge (W-1) was large enough that pore water pressure could have contributed to failure.

- The friction angles back-calculated for joint surfaces in Wedges W-1 through W-4 varied from 30 to 36 degrees. Based on the back analyses, an assumed friction angle of 30 degrees was selected for forward analyses.

In our discussions, OMR suggested that the modeling of W-1 for back analysis should be more detailed and incorporate local known conditions at the time of failure (proximal mining conditions, likely pore pressure conditions. etc.) The recollections of quarry personnel at the time of failure are discussed above. Those observations imply that blast disturbance was a likely factor in the 1998 failure of W-1. Mr. Woodbury stated that the movement of W-1 was triggered by mechanical excavation in the bench at the toe of the projected wedge immediately following blasting, in dry weather, with no observed groundwater seepage.

In their review comments, OMR assumed that ENGEO used the measurements from Station W-41 for the attitude of the joint (Set J5) bounding the west side of the wedge, and noted a 10 degree difference between the strike of the joint at Station W-41 (310 degrees) and the strike used for back-analysis (300 degrees). We note that the strike of the same joint at Station W-40 was measured at 300 degrees, and that this orientation was used for analysis, since it is closer to the actual location of the bench face that failed in 1998. .

In response to OMR comments, we prepared a detailed geologic model of Wedge W-1, (Figures 11 and 12) based on high-quality topography of the existing conditions obtained from the most recent overflight, collection of additional measurements of the orientations of the visible and accessible portions of the bounding joints. This model includes a slope geometry for the back-analysis that is based on the pre-failure topography from the 1998 flyover and on the description of the conditions at the time of failure as described by the quarry manager, Mr. Woodbury, who witnessed the failure. Based on this re-construction of the proximal conditions and on present day conditions in the quarry, we draw the following conclusions:

- Just prior to failure, most of the rock mass in W-1, including the portion nearest the excavation face, rested on two surfaces; a member of set J1 with a strike of approximately 300 and a dip of approximately 63 degrees, and a member of set J5 with a strike of 180 and a dip of 65 degrees.

- A third joint in set J5 with a strike of approximately 175 and a dip of 85 degrees formed a small portion of the uppermost part of W-1.

- The wedge hinge line did not daylight directly into the excavation, but passed below the surface approximately as shown on Figures 11 and 12. This geometry is consistent with the testimony of Mr. Woodbury who stated that the actual movement of the wedge occurred following blasting when material was being removed from the toe of the wedge.

- The movement of W-1 was triggered by a combination of blast fracturing and mechanical excavation that provided a path for a failure plane to reach the surface at the toe of the bench face at an elevation of approximately -140 msl. It is also likely that the integrity of the rock mass in the upper portions of W-1 was reduced by blast fracturing, given the condition of adjacent faces excavated at the same time.

- The failure occurred under dry conditions with no contribution from pore pressures.

In ENGEO's September 9, 2004, report, Wedge W-1 was modeled as a simple wedge case using the program SWEDGE©. We note that the three-dimensional geometry used in SWEDGE is limited to no more than two bounding joint planes, a tension crack and two planes representing the excavation face. In addition, SWEDGE can not analyze wedges that do not "daylight" into the face. In such a case, SWEDGE must be supplemented with a method that can model a compound failure surface of more than two bounding planes, passing through materials with differing shear strength. Accordingly, we have supplemented the SWEDGE analysis with a two-dimensional analysis of Cross Section 4 (Appendix B). We have also superimposed the previous geometry analyzed in ENGEO (2004) on Cross Section 4 for comparison purposes. The following assumptions were made in the two-dimensional analysis:

- The rock mass near the toe of the actively-mined slope within the wedge acted as a restraint to the movement of the wedge until it was disturbed by blasting and excavation. At the time of failure, we have assumed that this mass was partially or completely reduced to a

"disintegrated, poorly interlocked, heavily broken condition with a mixture of angular and rounded rock pieces" (Hoek 2000) with an assumed GSI of approximately 25.

- The cross section, drawn through the wedge hinge, approximates the most conservative case, where the largest width of restraining rock mass would have supported the rest of the wedge.

- The bounding joints were smooth surfaces approximated by friction-only shear strength with no cohesion contributed from joint roughness.

If it is assumed that the failure came to the surface at the toe of the bench at -140 msl, the back-calculated shear strength for the bounding joints is 35 degrees. If it is assumed that the failure surface exited the face at the bench at -100 msl, the back-calculated joint shear strength is 32.5 degrees. The slope stability outputs from two-dimensional analyses are presented in Appendix B along with the previous SWEDGE output. From the above analyses, we conclude the following regarding our previous and current wedge back-analysis findings:

- The previously reported back-analyzed frictional shear strength for the bounding joints at W-1 was conservative and appears to be at the lower bound of permissible assumptions for back-analyzed frictional strength for the joint surfaces.

- The previously-reported geometry used for back-analysis of W-1 in SWEDGE is a relatively close approximation of the more complex geometry depicted in this report, as demonstrated by the graphical output from the 2004 ENGEO back-analysis that is superimposed on Cross Section 4.

- The back-analyzed joint friction angles show a close similarity with the values reported by GTU (2000) from Yerba Buena Island. We note that average of the initial, peak joint strength values reported by GTU (36 degrees) is significantly higher than the assumed friction angle used for our analyses (30 degrees). In fact, an assumed frictional strength of 30 degrees is close to the average disturbed or "final" frictional joint strength of 29 degrees (neglecting any cohesion) reported by GTU. In our opinion, an assumed frictional joint shear strength of 30 degrees is a conservative, lower-bound value that is well-supported by both back-analysis and testing.

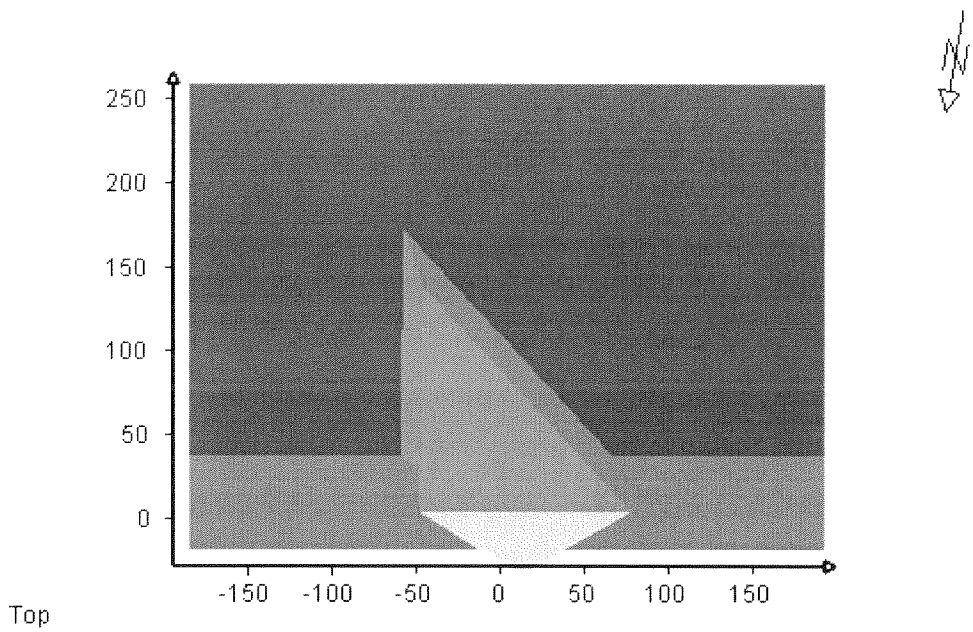
After implementation of the reclamation plan the risk of wedge failures like the one that occurred in 1998 will be greatly reduced by the following factors:

- The finished faces excavated using improved blasting practices will expose rock that is less disturbed, and therefore, stronger than the current exposures on the south face.

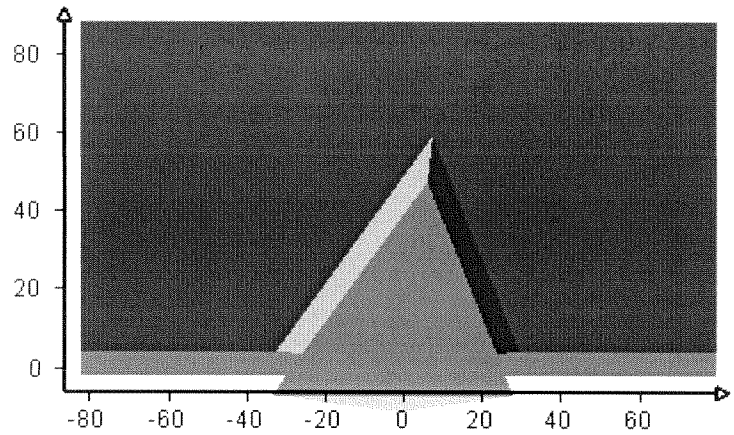
- Geologic inspections will identify possible unstable areas in the pit walls as mining proceeds. Recommendations for mitigation of significant unstable areas will be provided during mining and prior to reclamation.

- Flooding of the pit will equalize pore pressures and increase resisting forces on the quarry walls.

The back-analyzed wedges are illustrated below.

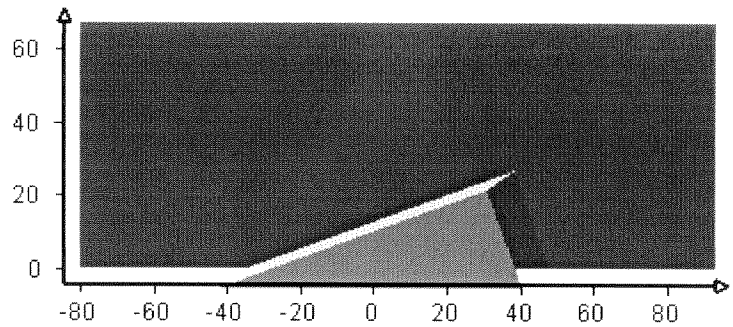


WEDGE 1 (EXISTING FAILURE, SOUTH FACE, TOP VIEW)



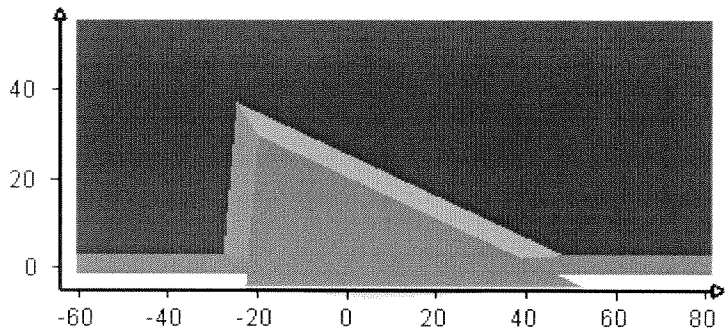
Top

WEDGE 2 (SOUTH FACE TOP VIEW)



Top

WEDGE 3 (NORTH FACE TOP VIEW)



Top

WEDGE 4 (SOUTH FACE TOP VIEW)

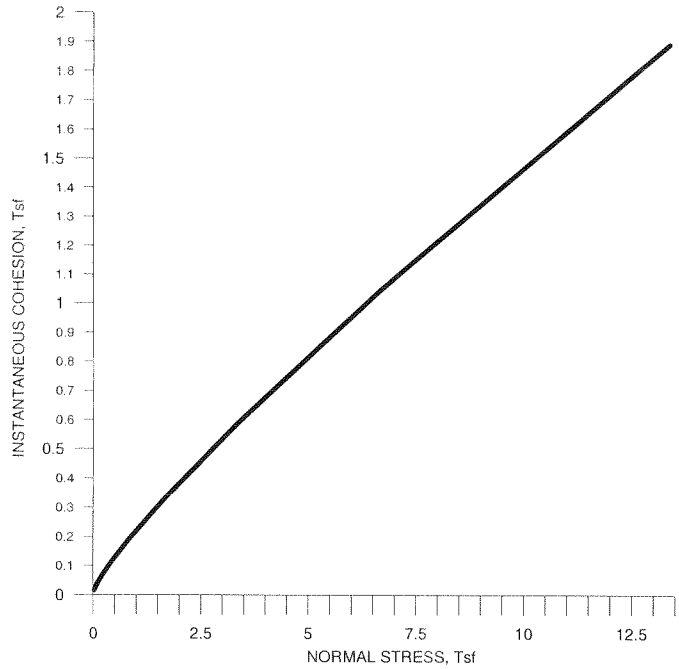
Selection of Joint Shear Strength Parameters

As described above, four existing small- to-medium-sized wedge-failures were back-analyzed to estimate a lower-bound frictional shear strength (phi) for discontinuities. The results of back-analyses were compared to direct-shear test results obtained from Yerba Buena Island, as described above.

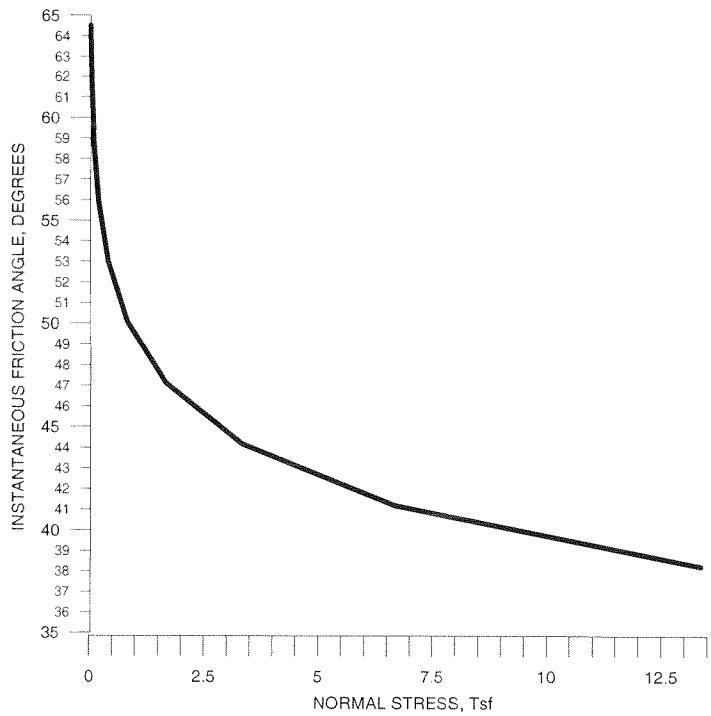
In forward analyses, the Mohr-Columb shear strength was estimated from the Barton and Bandis (1990) criterion by calculating the instantaneous cohesion and friction for the appropriate normal stress as described in Hoek (2000). Conservative values were assumed for the calculation, including:

- A base friction angle of 30 degrees.
- A Joint Compressive Strength (JCS) of 3,500 psi (one-half the UCS). This assumption was based on the fact that the wall rock adjacent to some joint surfaces was observed to be affected by shearing and/or alteration or was filled with calcite-cemented rock breccia. It should be noted that the assumed UCS is also conservative, based on the test data. However, we feel that this assumption is appropriate for this preliminary study.
- A JRC of 10, which is in the lower range of observed joint surface conditions.
- The value of instantaneous friction angle used for this study was capped at 45 degrees. The Barton and Bandis (1990) criterion theoretically calculates instantaneous friction angles of over 60 degrees; however, we limited the maximum friction angle used in calculation to no more than 15 degrees higher than the base friction angle.

The calculated plots of normal stress versus instantaneous cohesion and normal stress versus friction angle are presented below:



INSTANTANEOUS COHESION



INSTANTANEOUS FRICTION ANGLE

As noted above, many joints in the Set J5 were observed to offset bedding and are therefore faults. However, the faulted joint surfaces are typically curvelinear and are "healed" and re-cemented with calcite or filled with cemented breccia. No weak clay fillings were observed. Three of the back-analyzed wedges (W-1, W-2 and W-3) included a member of Set J5. W-1 and W-2 included joints that have cemented breccia fills. We therefore feel that the effects of faulting on joint shear strength have been adequately accounted for by back-analysis and assumption of lower-bound values for the base friction angle and cohesion.

Forward Analysis of Wedges

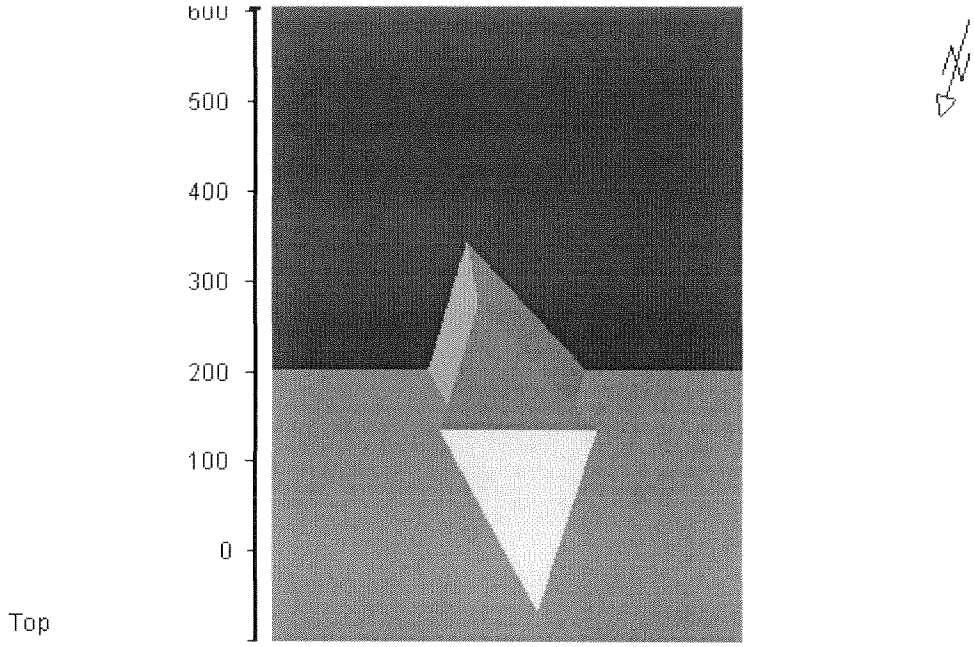
Forward analysis of possible structurally-controlled slope failures was performed based on sections of kinematically adverse wedges from stereonet analyses. Three wedge cases were selected from the north face and seven cases were selected from the south face. Two of the south face cases included the geometry of back-analyzed wedges (Wedges 1 and 2) scaled to the proposed maximum pit depth. Another case from the south face (Case 5) approximates a plane failure along bedding. For the purpose of analysis, we scaled the selected wedges up to the maximum size possible based on the proposed final pit configuration (60-degree slope inclination and 350-foot vertical height). This is a conservative assumption, since all of the existing wedge failures observed to date have occurred as local failures in steep inter-bench faces, with the exception of W-1 which had a height of approximately 140 feet. The analyses also assume the worst case spatial relationship in which the line of intersection of the wedge "daylights" precisely at the toe of the overall slope without a bridge of intact rock. The analyzed cases are identified on Figures 6 through 9

Forward analyses of wedges was carried out using the program Swedge© and instantaneous cohesion and friction, estimated as described above. The normal stress for each joint surface was estimated based on the SWEDGE © output, as presented in Appendix B. For flooded conditions,

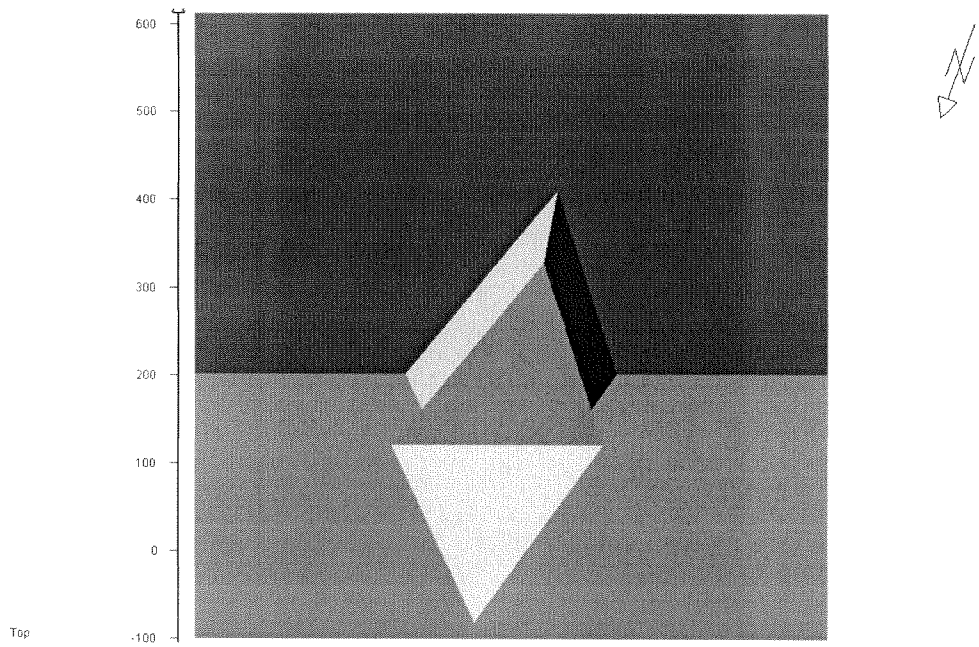
joint shear strengths were reduced by using the buoyant unit weight for the rock in the Barton-Bandis calculation.

Static slope stability analyses were performed for the proposed maximum pit depth in the dry condition, and in the post-flooded condition. Pseudo-static analyses were performed for the flooded condition. The applied pseudo-static coefficient was 0.19, as determined by the method of Ashford and Sitar (2002). For the flooded, seismic loading case, the analyses used the reduced buoyant joint shear strength, but included the full, non-buoyant weight of the wedge in the calculation. Assumed minimum acceptable factors of safety (FS) were 1.5, 1.1 and 1.15 for the proposed final reclamation plan static condition (pit full of water), the maximum pit-depth mining static conditions (no water in pit) and the proposed final reclamation plan seismic loading condition (pit full of water), respectively.

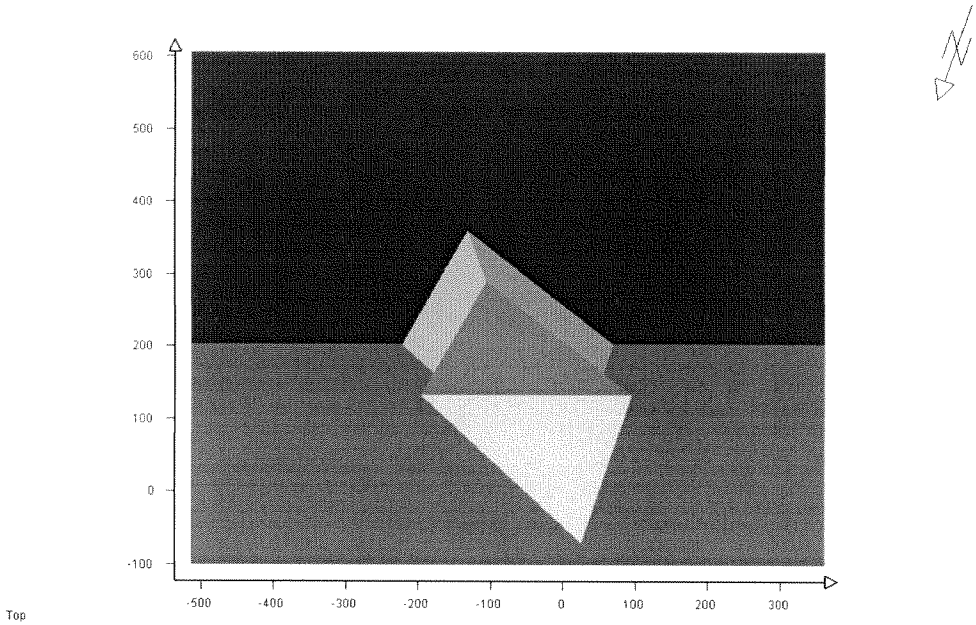
For the open-pit dry condition, it was assumed that discontinuities were 50 percent filled with pore water for all wedges except Case 1 at the north wall. Examination of cross sections of the wedges showed that the projected wedge surfaces were less than 50 percent below the projected water surfaces depicted the Cross Sections, with the exception of Case 1 at the north wall. Case 1 is the largest of the analyzed wedges, and it was assumed that the fractures were 100 percent filled with water. Under flooded pit conditions, where no un-balanced pore pressures will be present, the wedges were analyzed with the buoyant unit weight of the rock (165 pcf – 62.4 pcf) and the pore pressures set to zero. The analyzed wedges are depicted graphically below.



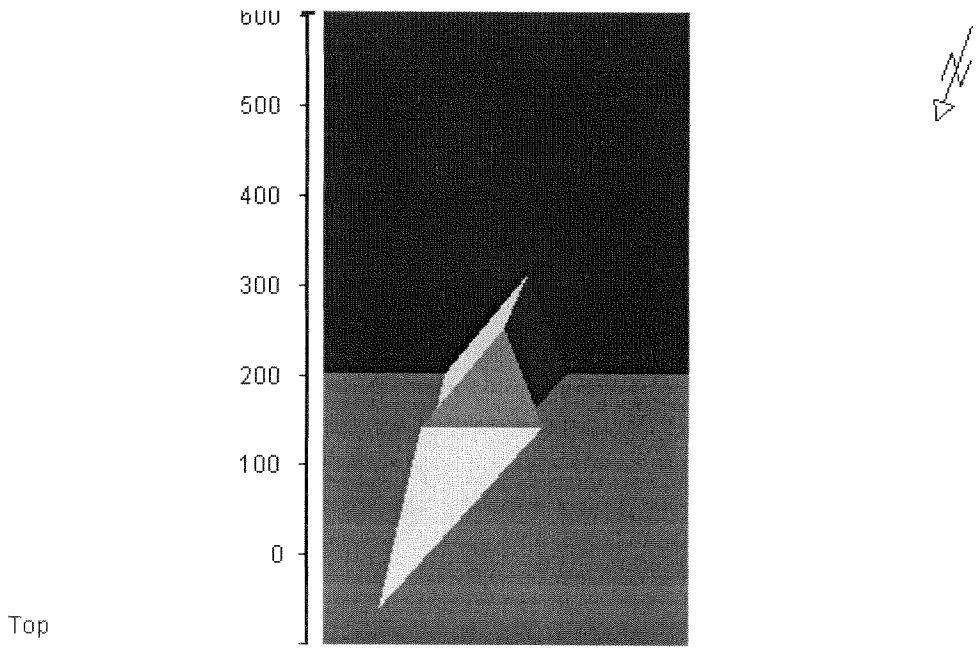
WEDGE 1 SCALED TO 350-FOOT HEIGHT
(TOP VIEW)



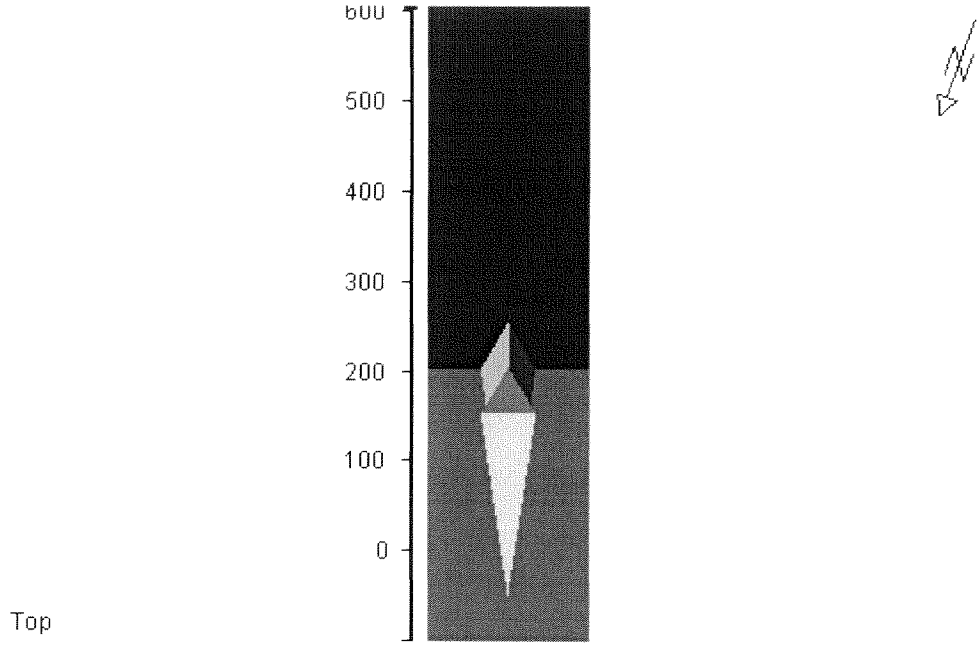
WEDGE 2 SCALED TO 350-FOOT HEIGHT
(TOP VIEW)



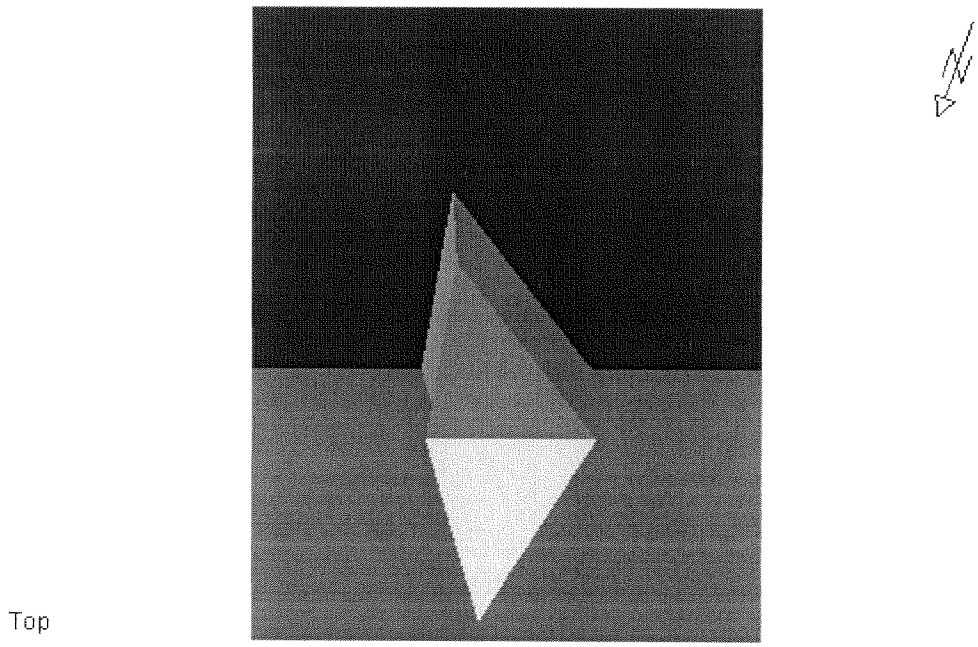
CASE 1 SOUTH WALL
(TOP VIEW)



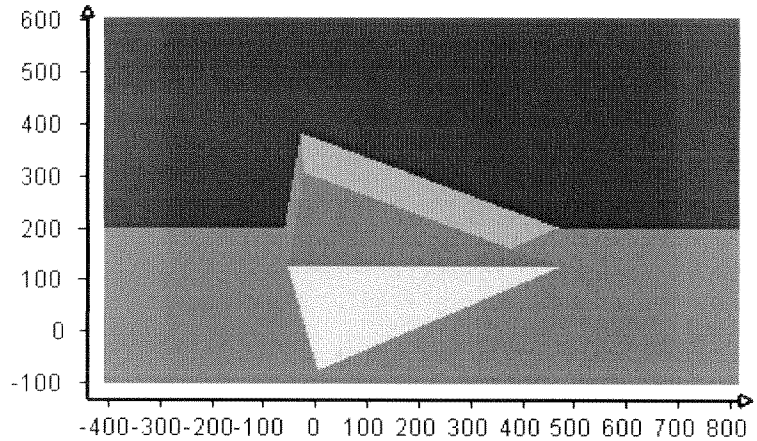
CASE 2 SOUTH WALL
(TOP VIEW)



CASE 3 SOUTH WALL
(TOP VIEW)

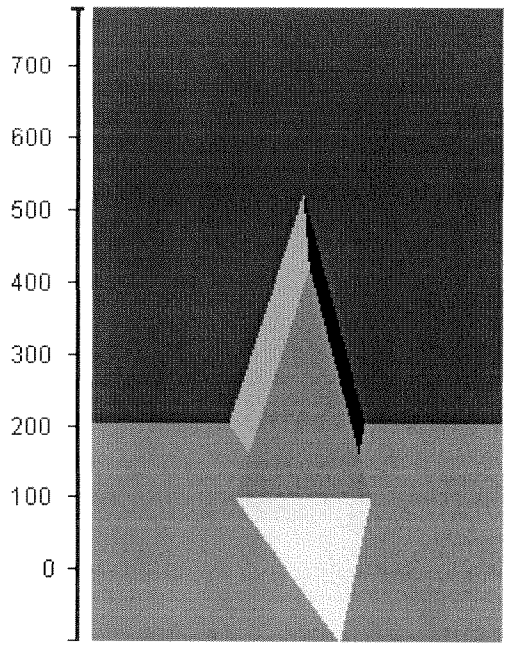


CASE 4 SOUTH WALL
(TOP VIEW)



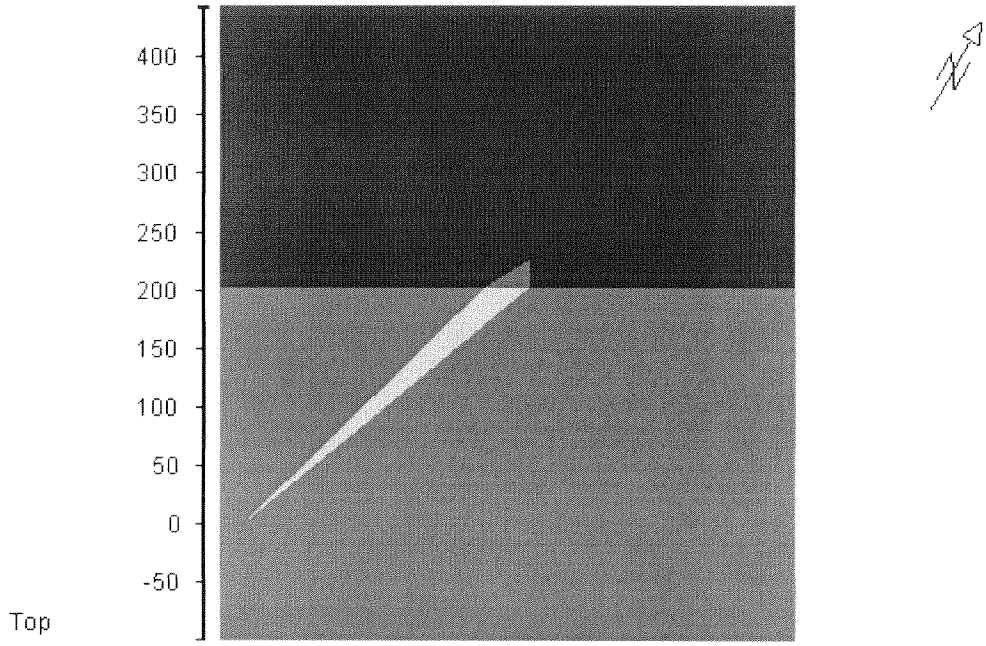
Top

CASE 5 SOUTH WALL
(TOP VIEW)

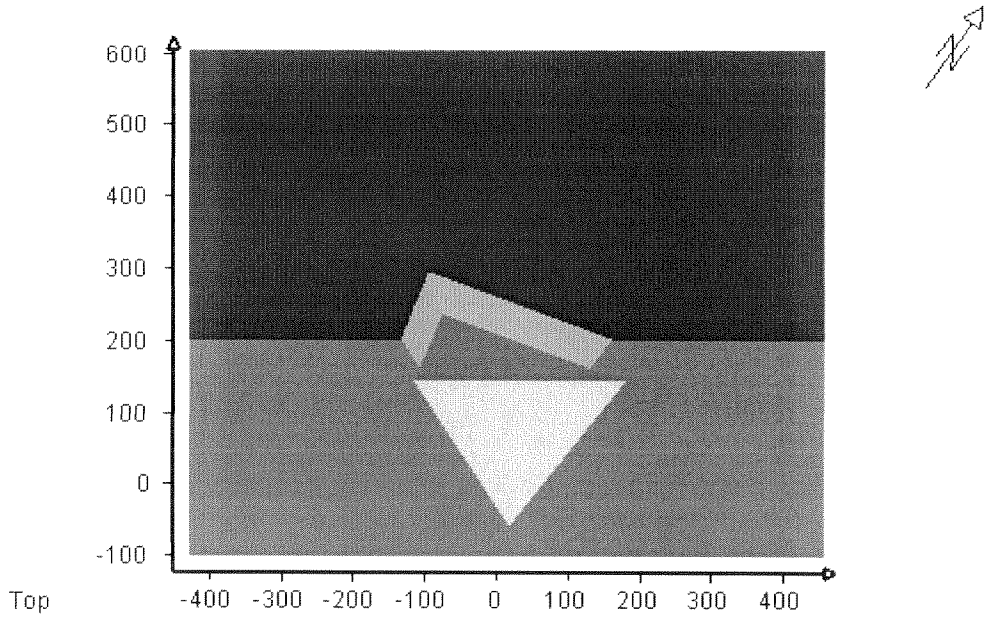


Top

CASE 1 NORTH WALL
(TOP VIEW)



CASE 2 NORTH WALL
(TOP VIEW)



CASE 3 NORTH WALL
(TOP VIEW)

TABLE 4
Results of Wedge Slope Stability Analyses

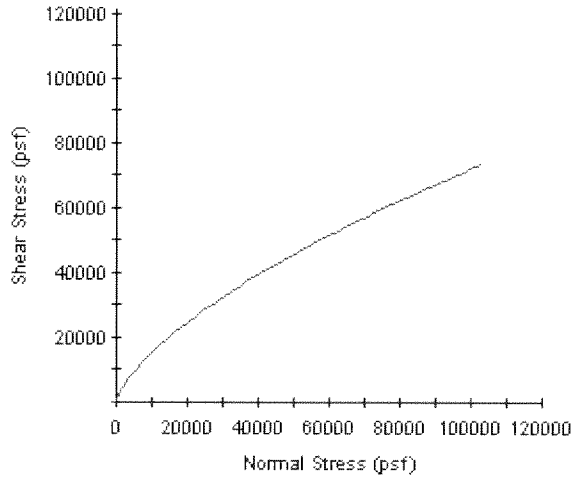
Analyzed wedge	Condition	Min. Acceptable F.S.	Calculated F.S.	Pore Pressure Assumptions
Wedge 1, Scaled to 350-Foot Height	Proposed, Max Pit, Static	1.1	1.67	Fractures 50% Filled
	Proposed, Flooded, Static	1.5	1.95	Equalized
	Proposed, Flooded, Pseudo-Static	1.15	1.24	Equalized
Wedge 2 Scaled to 350-Foot Height	Proposed, Max Pit, Static	1.1	1.94	Fractures 50% Filled
	Proposed, Flooded, Static	1.5	2.20	Equalized
	Proposed, Flooded, Pseudo-Static	1.15	1.44	Equalized
Case 1 South Wall	Proposed, Max Pit Static	1.1	1.47	Fractures 50% Filled
	Proposed, Flooded, Static	1.5	1.66	Equalized
	Proposed, Flooded, Pseudo-Static	1.15	1.21	Equalized
Case 2 South Wall	Proposed, Max Pit Static	1.1	1.57	Fractures 50% Filled
	Proposed, Flooded, Static	1.5	1.86	Equalized
	Proposed, Flooded, Pseudo-Static	1.15	1.21	Equalized
Case 3 South Wall	Proposed, Max Pit Static	1.1	1.91	Fractures 50% Filled
	Proposed, Flooded, Static	1.5	3.1	Equalized
	Proposed, Flooded, Pseudo-Static	1.15	1.73	Equalized
Case 4 South Wall	Proposed, Max Pit Static	1.1	2.25	Fractures 50% Filled
	Proposed, Flooded, Static	1.5	2.24	Equalized
	Proposed, Flooded, Pseudo-Static	1.15	1.50	Equalized
Case 5 South Wall	Proposed, Max Pit Static	1.1	1.45	Fractures 50% Filled
	Proposed, Flooded, Static	1.5	1.64	Equalized
	Proposed, Flooded, Pseudo-Static	1.15	1.19	Equalized
Case 1 North Wall	Proposed, Max Pit Static	1.1	2.09	Fractures 100% Filled
	Proposed, Flooded, Static	1.5	3.78	Equalized
	Proposed, Flooded, Pseudo-Static	1.15	2.45	Equalized
Case 2 North Wall	Proposed, Max Pit Static	1.1	1.62	None
	Proposed, Flooded, Static	1.5	1.63	Equalized
	Proposed, Flooded, Pseudo-Static	1.15	1.26	Equalized
Case 3 North Wall	Proposed, Max Pit Static	1.1	1.17	Fractures 50% Filled
	Proposed, Flooded, Static	1.5	1.43	Equalized
	Proposed, Flooded, Pseudo-Static	1.15	1.15	Equalized

Circular Failure Analyses of Quarry Slopes

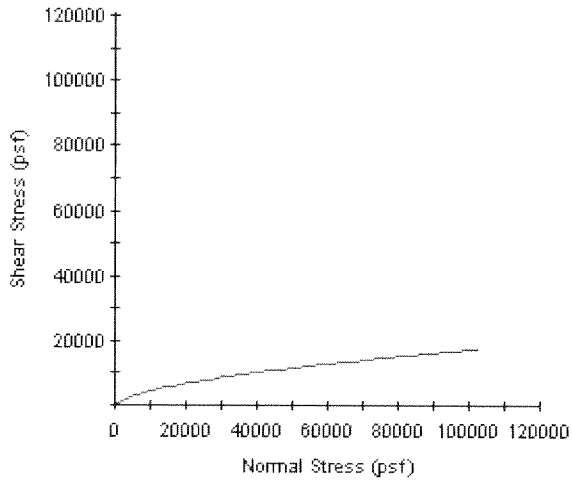
Limit equilibrium slope stability analyses were carried with the program SLIDE© Version 5.010 by Rocscience Inc. with SWEDGE© also by Rocscience, Inc. The program Slide incorporates the generalized Hoek-Brown failure criterion as described in Hoek (2000). Circular failure analyses were carried out using the modified Hoek-Brown method incorporated into the SLIDE program. The input criteria used for the analysis were based on conservative assumptions as described below:

- The GSI for the un-weathered graywacke was estimated to range from 30 to 60; a GSI of 50 was used for calculation.
- The GSI for weathered greywacke exposed at the south brow was assumed to be 30.
- Conservative value of 7,000 psi 3,472 psi and 3,000 psi were used for the UCS of unweathered greywacke weathered greywacke and shale, respectively.
- The rock disturbance factor (D) was assumed to be equivalent to heavy production blasting (1.0). As previously described, this assumption is conservative because the slope stability model assumes uniform rock disturbance throughout the rock mass, when in reality, the influence of blasting for an excavation of the size of the subject quarry is not likely to extend very far from the exposed face. In addition, improved blasting procedures are likely to reduce disturbance in the final quarry faces. For comparison purposes, the south portion of Section 2 was analyzed with D = 0.7, as presented below.

The shear strength functions used for analyses (exported from SLIDE), are depicted below. The range of normal stresses applicable to this study ranges from 0 psf to approximately 40,000 psf. A detailed description of shear strength parameters used for each analysis is depicted on the slope stability outputs presented in Appendix B.



META-GRAYWACKE, GSI=50 (MODIFIED HOEK-BROWN)



META-SHALE, GSI=25 (HOEK-BROWN)

Analyzed slope geometries included the existing pit, the proposed maximum pit depth with no water, and the proposed maximum pit depth with the pit flooded. Analyses were performed for all of the above geometries under static loading conditions; pseudo-static loading was evaluated for the post-mining condition. The applied pseudo-static coefficient was 0.15, as described above. Assumed minimum acceptable factors of safety (FS) were 1.5, 1.1 and 1.15 for the proposed final reclamation plan static condition (pit full of water), the maximum pit-depth mining static condition (no water in pit) and the proposed final reclamation plan seismic loading condition (pit full of water), respectively. Analyses were carried out for Cross Sections 1, 2 and 3, located as shown on Figure 5. It was assumed that the areas peripheral to the pit would be graded to accommodate housing construction as depicted in APR 04. The proposed finished grades, are depicted on the slope stability outputs in Appendix B. The results of circular failure surface analyses are summarized in Table 5 below.

TABLE 5
Results of Circular Slope Stability Analyses

Section	Condition	Assumed Min. Acceptable F.S.	Min. Calculated F.S.
Section 1 North Face	Existing, Static	1.1	2.10
	Proposed, Max Pit, Static	1.1	1.60
	Proposed, Flooded, Static	1.5	2.16
	Proposed, Flooded, Pseudo-Static	1.15	1.40
Section 1 South Face	Existing, Static	1.1	2.15
	Proposed, Max Pit, Static	1.1	1.64
	Proposed, Flooded, Static	1.5	1.99
	Proposed, Flooded, Pseudo-Static	1.15	1.35
Section 2 North Face	Existing, Static	1.1	2.2
	Proposed, Max Pit Static	1.1	1.17
	Proposed, Flooded, Static	1.5	1.86
	Proposed, Flooded, Pseudo-Static	1.15	1.16
Section 2 South Face	Existing, Static	1.1	2.0
	Proposed, Max Pit Static	1.1	1.14
	Proposed, Flooded, Static	1.5	1.75
	Proposed, Flooded, Pseudo-Static	1.15	1.19

TABLE 5
Results of Circular Slope Stability Analyses

Section	Condition	Assumed Min. Acceptable F.S.	Min. Calculated F.S.
Section 2 South Face D = 0.7	Proposed, Max Pit Static	1.1	1.46
	Proposed, Flooded, Static	1.5	2.26
	Proposed, Flooded, Pseudo-Static	1.15	1.55
Section 3	Proposed, Max Pit Static	1.1	1.20
	Proposed, Flooded, Static	1.5	1.81
	Proposed, Flooded, Pseudo-Static	1.15	1.22

The most critical analyzed case appears to be the maximum pit-depth mining condition (seismic loading was not evaluated for this condition). Calculated factors of safety for this case ranged from 1.14 to 1.64. All calculated factors of safety exceeded minimum acceptable values. It is worth noting that the proposed mining sump at elevation -400 msl (depicted on Cross Section 1) does not adversely affect the stability of the pit walls, due to the horizontal distance between the rim of the sump and the base of the proposed pit walls at -350 msl.

Newmark Analysis

A Newmark seismic deformation analysis was performed on the north quarry wall at Section 2. The section was chosen based on calculation of the yield acceleration (F.S. =1.0) for both the north and south walls of Sections 1 and 2. The lowest yield acceleration (0.192) was calculated for north face on Section 2. A Newmark-type deformation analysis assumes that a well-defined slip surface develops, the material is rigid-perfectly plastic, and that the material does not lose strength during strong shaking. The yield acceleration is the governing input parameter in the analysis. The yield acceleration is the seismic coefficient that results in incipient failure (factor of safety, FS, equal to 1.0) in a pseudostatic stability analysis. Based on the probabilistic analysis described above, the design peak ground acceleration was assumed to be 0.60g.

Using a conservative value of the maximum acceleration ratio, the calculated seismic displacements are 0.2 to 0.9 cm which is considered to be negligible (Makdisi & Seed, 1978; SCEC, 2002). It is important to note that seismic displacements calculated based on Newmark-type analyses are not considered to be predictions of actual expected ground displacement, but are in effect intended to evaluate the potential for significant seismic slope failure. The calculated displacements would indicate that large-scale seismic slope deformation would be unlikely for the post reclamation slopes with a flooded pit. Supporting calculations are included in Appendix B.

CONCLUSIONS

- Analyses presented in this report show that the pit configuration in ARP 04 has resulted in increased stability in all cases. As the attached data and analyses demonstrate, the conclusions and recommendations of the September 9, 2004 ENGEO report have been confirmed by this supplemental report. Therefore, based on the results of our field mapping, subsurface exploration, data review and analyses, it appears that ARP 04 can be implemented as proposed.

- The deposit currently mined at the quarry consists of hard graywacke sandstone of the Franciscan Assemblage. In the quarry pit area, the mineable deposit continues to a depth of at least 550 feet below mean sea level. Mineable sandstone is also present at depth below the South Hill area, below a weathered rock deposit. The geotechnical characteristics of the rock mass, including discontinuities, groundwater conditions and intact rock strength were evaluated based on the best available information from site-specific geologic mapping, previous detailed rock core logs, published Bay Area geologic and geotechnical information using analysis methods appropriate for rock slope engineering. Rock slope stability was evaluated for the existing pit, the maximum proposed mining depth and for the post-mining, flooded pit condition. The proposed deepened pit was analyzed for static conditions and for earthquake conditions (pseudo-static loading, topographic seismic amplification effects and Newmark deformation analysis).

- The calculated static and pseudo-static factors of safety for quarry pit slopes are within generally acceptable limits for mining at the proposed final pit elevation of -350 feet msl and for later adjacent residential, marina and commercial construction, assuming that the pit is flooded after mining. Topographic seismic amplification effects are not expected to adversely affect post-mining reclamation and second use plans, assuming that the recommendations of this report are adhered to. Seismic slope deformation calculated for quarry pit slopes based on a Newmark analysis is estimated to be negligible.

- Prior to the final design of proposed improvements, the findings of this report should be re-evaluated based on exposed post-mining conditions within the pit. The location of proposed habitable structures and critical facilities such as lifeline roads and utilities with respect to the top finished pit reclamation slopes should be based on detailed post-mining studies.

- Conservative geotechnical assumptions were used throughout the analyses presented in this report, with the intent to provide a geologic model with very conservative factors of safety for planning purposes. Recommended future studies will include geologic mapping as new exposures are created during mining, additional rock shear strength testing, and monitoring

of groundwater levels. Based on additional data, it may be possible to justify higher rock mass shear strengths and locally steeper slope inclinations if appropriate.

RECOMMENDATIONS

A. Supplemental Geotechnical Pit Observations

- The pit exposures should be observed and evaluated as mining proceeds by a qualified engineering geologist and/or mining engineer. The purpose of observation during mining would be to identify possible adverse rock structure as excavations proceed, so that the quarry operations can avoid undesirable slope failures in critical improvements such as access ramps or quarry brow improvements. Since much of the proposed quarrying area is currently not exposed, supplemental observations will be carried out to adequately address geologic conditions as they are revealed by mining.

- As a minimum, a thorough re-evaluation of excavated slopes should be performed near the conclusion of the mining operations so that the proposed post-reclamation conversion to Second Uses contained in ARP82 and ARP04 can be re-evaluated based on revealed conditions.

B. Groundwater Monitoring

- We recommend that piezometers be installed around the existing pit margin during the balance of the mining process to measure the elevation of the existing piezometric surface or surfaces and to confirm assumptions about pore water pressures made for this analysis as monitoring proceeds.

- The actual configuration of the piezometer array should be determined based on the final proposed pit configuration and on proposed planning of quarry operations to allow optimum placement of instruments and to avoid conflicts with future operations.

C. Slope Monitoring

- We recommend that a network of survey monitoring points be established at selected locations around the pit brow and on selected benches. The survey net should be monitored monthly during periods of active mining or at more frequent intervals if appropriate to monitor suspected areas of movement.

D. Recommended Future Studies

- We recommend that the quarry pit slope stability characterization presented in this report be supplemented at the conclusion of mining, and prior to conversion to the Second Uses contained in ARP04, with a comprehensive re-evaluation of quarry slope stability based on the results of on-site geotechnical pit observations made during mining, groundwater monitoring, slope monitoring, and a program of laboratory testing of on-site materials. An appropriate testing program should, as a minimum, include unconfined compression tests, triaxial testing, and direct shear tests of joint surfaces.

- At the conclusion of mining activities, a qualified engineering geologist or mining engineer should prepare a revised geologic map of the pit and provide supplemental recommendations for implementation of the proposed reclamation plan. If required, supplemental rock slope engineering recommendations should be provided to maintain acceptable factors of safety for proposed land uses.

- The preliminary descriptions presented in this report of subsurface soil conditions outside the pit area should be confirmed with a comprehensive geotechnical site exploration. We anticipate that such an exploration will include geotechnical borings and sampling of site soils, laboratory testing and analyses of collected data focused on providing design-level recommendations to mitigate the geotechnical development constraints identified in this study.

- The design-level geotechnical report should include detailed and site-specific recommendations for grading, mitigation of compressible and liquefiable soils, slope stability analyses, recommendations for appropriate foundations for residential and commercial structures and geotechnical recommendations for construction of underground utilities and surface drainage improvements. In concert with the final design of site grading plans, ENGEO typically provides the client with a detailed corrective grading plan showing the locations of keyways slope stabilization earthwork, subdrains, and the limits and depth of removals intended to mitigate geotechnical condition such as compressible or unsuitable soils. Corrective grading plans for this site would also include depictions of the recommended limits of geotechnical remediation measures such as surcharge fills, wick drains, structural earth retention and ground improvement.

- All recommended geotechnical remediation measures should be coordinated with the project civil engineer so that they can be properly incorporated in the final grading plans.

E. Quarry Slope Design

- Within the quarry pit, the average (toe to top) slope inclination should not exceed 60 degrees for a maximum vertical height of 350 feet, as depicted on Figure 15.

- Minimum 30-foot-wide safety benches should be constructed at maximum 90-foot vertical intervals.

- In general, the inclination of inter-bench faces should be maintained at less than 75 degrees where possible. The recommended safety bench spacing and width are depicted on Figure 14. Locally, inter-bench face inclinations will be influenced by splitting along pre-existing rock discontinuities, but overhanging faces should be avoided wherever possible. The bench and slope angle recommendations presented here have been incorporated into ARP 04.

- The quarry access ramp placement required to deepen the quarry has been configured to minimize excavation at the south face and create a buttressing effect to the slopes at the south side of the quarry.

- Quarry pit design should consider the potential effect of large-scale horizontal curvature of pit walls on slope stability. In general, convex-inward horizontal curves in quarry slopes should be avoided. Concave inward slopes offer some degree of increased confinement by "arching" of the rock mass between discontinuities, and effectively decrease the area of free face available for kinematically possible failure geometries. Convex-inward slopes can actually contribute to potential instability, since lateral confinement is reduced and the area of the kinematically-available free face is effectively increased. The concave-inward slopes contained in the plan to deepen the quarry and reflected in ARP 04 is more appropriate for stability than the configuration shown in ARP82.

F. Slope Stability Mitigation Options for Mining

- The periodic geotechnical inspections recommended above should include evaluation of mining faces for potentially unstable blocks. Localized face failures are an expected part of surface mining, and the location and potential size of unstable blocks can be evaluated during periodic inspections as mining proceeds. If it appears that a critical facility such as the access ramp could be threatened by a potential block failure, the geotechnical engineer could recommend appropriate corrective action such as the installation of rock bolts, or local modification of mining excavations to increase stability.

- The large-scale stability of the quarry walls should be periodically evaluated by the geotechnical engineer based on the results of monitoring of slope performance, groundwater levels, and geotechnical inspection of mining exposures. If unacceptable slope performance is detected, it will be possible to implement several possible mitigation measures as described below. The actual recommended mitigation measures should be based on site-specific evaluations and should be implemented based on an appropriate evaluation of risk and cost.

- Mitigation measures should be employed if adverse groundwater conditions are encountered (unacceptably high pore pressures or excessive seepage, etc.) Mitigation measures could include horizontal drains, extraction wells, slurry walls, etc..

- If unacceptable levels of mining-concurrent slope deformation are encountered, mining activities can be modified to improve stability. At the quarry brow, stockpiles of products, quarry waste piles or areas of overburden can be excavated and moved to reduce driving forces. In the pit, bench configurations can be modified by "stepping out" or increasing bench width, effective flattening the mining slope angle.

- At the south quarry brow, it is anticipated that the final slopes will locally expose quarry fills and areas of native soils and weathered rock. The anticipated extent of soils and weaker materials in the proposed face is presented in Figure 13. Figure 14 presents options for mitigation, including construction of a sheet pile wall or an engineered fill buttress. Both options would allow the quarry limits depicted in SRRQ's mining plan to be preserved.

G. Location of Improvements Near Quarry Pit Slopes

- Prior to the final design of proposed improvements, the findings of this report should be re-evaluated based on exposed post-mining conditions within the pit.

- The location of Second Use structures and critical facilities such as lifeline roads and utilities with respect to the top finished pit reclamation slopes should be based on the results of the recommended detailed post-mining studies.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, contractors, buyers, architects, engineers, and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The conclusions and recommendations contained in this report are solely professional opinions.

The professional staff of ENGEO Incorporated strives to perform its services in a proper and professional manner with reasonable care and competence but is not infallible. There are risks of earth movement and property damages inherent in mining operations land development. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our work.

This report is based upon field and other conditions discovered at the time of preparation of ENGEO's work. This document must not be subject to unauthorized reuse, that is, reuse without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time. Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's work. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes as construction activities proceed. If ENGEO's scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims, including, but not limited to claims arising from or resulting from the performance of such services by other persons or entities, and any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

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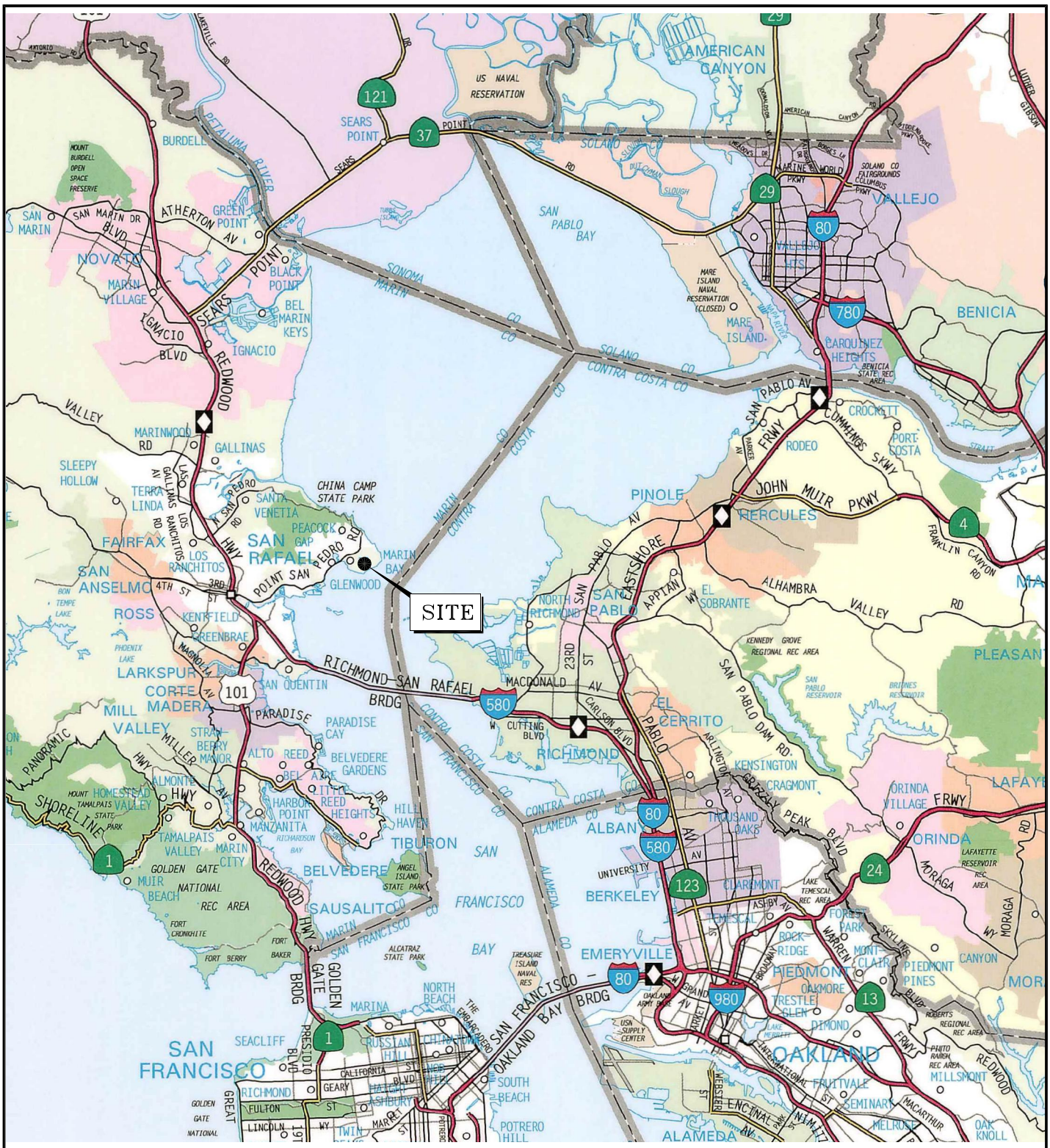
LIST OF FIGURES



BASE MAP SOURCE: THOMAS BROTHERS

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

SITE VICINITY MAP
SAN RAFAEL QUARRY
SAN RAFAEL, CALIFORNIA

PROJECT NO.: 6261.1.003.01	FIGURE NO. 1
DATE: APRIL 2005	
DRAWN BY: PC	CHECKED BY: PJS

ORIGINAL FIGURE PRINTED IN COLOR

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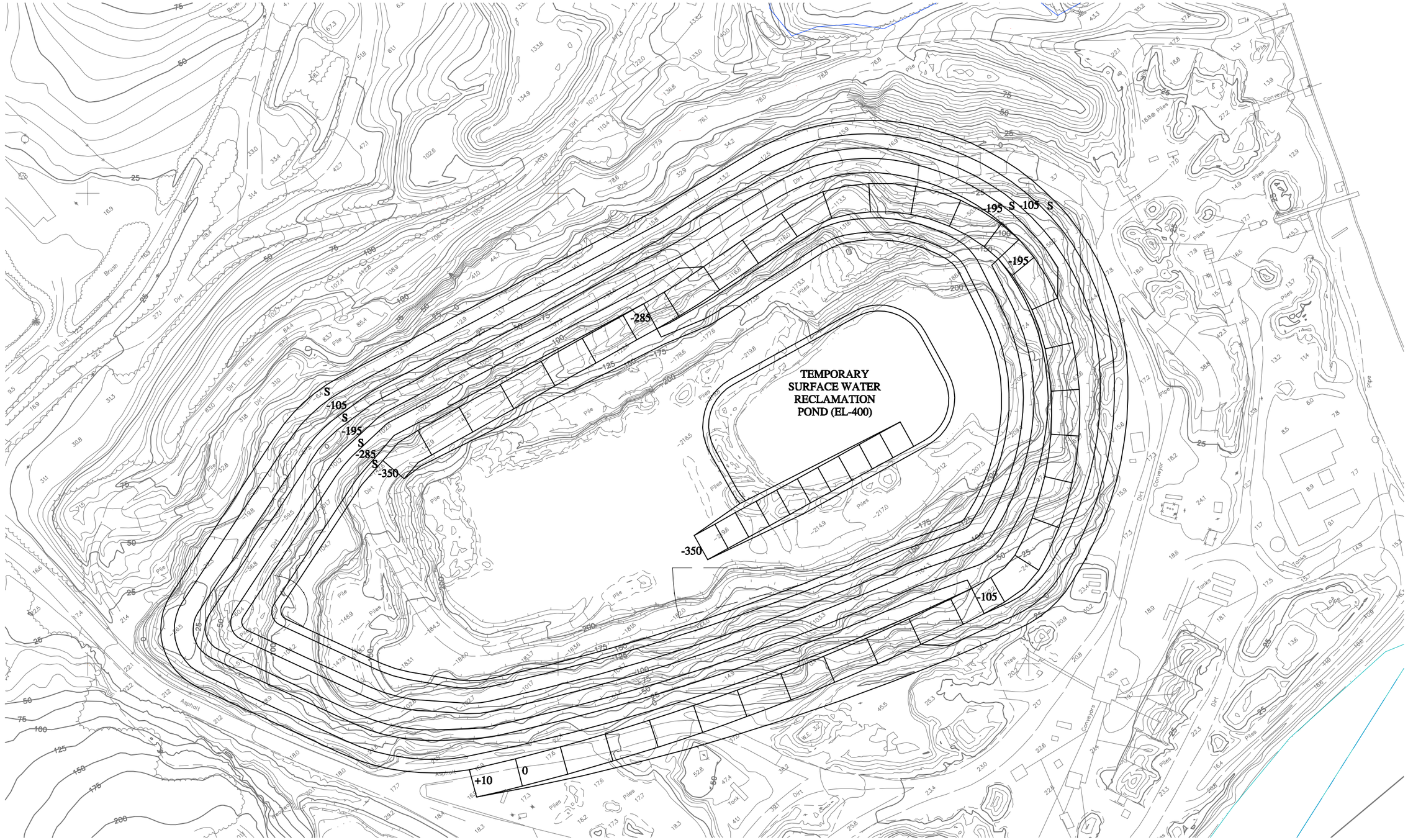

N

0 FEET 200
0 METERS 100

BASE MAP SOURCE: TOWILL SURVEY AND MAPPING, SAN FRANCISCO, CALIFORNIA

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

PROPOSED MINING PLAN
SAN RAFAEL QUARRY
SAN RAFAEL, CALIFORNIA

PROJECT NO.: 6261.1.003.01		FIGURE NO.
DATE: APRIL 2005		2
DRAWN BY: PC	CHECKED BY: PJS	

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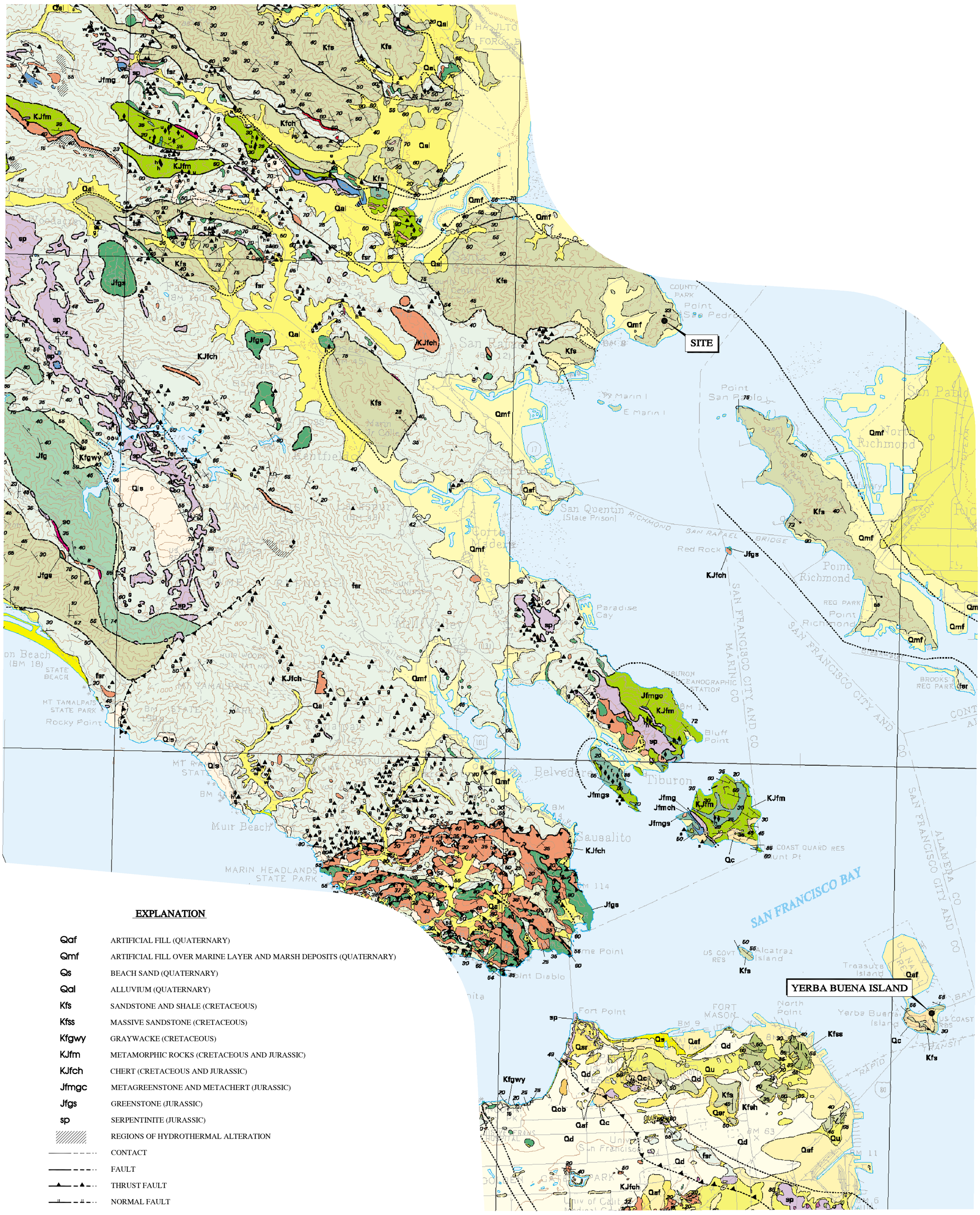

EXPLANATION

Qaf	ARTIFICIAL FILL (QUATERNARY)
Qmf	ARTIFICIAL FILL OVER MARINE LAYER AND MARSH DEPOSITS (QUATERNARY)
Qs	BEACH SAND (QUATERNARY)
Qal	ALLUVIUM (QUATERNARY)
Kfs	SANDSTONE AND SHALE (CRETACEOUS)
Kfss	MASSIVE SANDSTONE (CRETACEOUS)
Kfgwy	GRAYWACKE (CRETACEOUS)
KJfm	METAMORPHIC ROCKS (CRETACEOUS AND JURASSIC)
KJfch	CHERT (CRETACEOUS AND JURASSIC)
Jfmgc	METAGREENSTONE AND METACHERT (JURASSIC)
Jfgs	GREENSTONE (JURASSIC)
sp	SERPENTINITE (JURASSIC)
	REGIONS OF HYDROTHERMAL ALTERATION
	CONTACT
	FAULT
	THRUST FAULT
	NORMAL FAULT
▲	HIGH-GRADE MELANGE BLOCK
♦	LOW GRADE MELANGE BLOCK
	STRIKE AND DIP OF STRATA

INCLINED VERTICAL OVERTURNED

0 FEET 6000
0 METERS 3000

N

BASE MAP SOURCE: USGS, M.C. BLAKE JR., R.W. GRAYMER, D.L. JONES, 2000

ENGEO INCORPORATED
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REGIONAL GEOLOGY MAP
SAN RAFAEL QUARRY
SAN RAFAEL, CALIFORNIA

PROJECT NO.: 6261.1.003.03	FIGURE NO. 3
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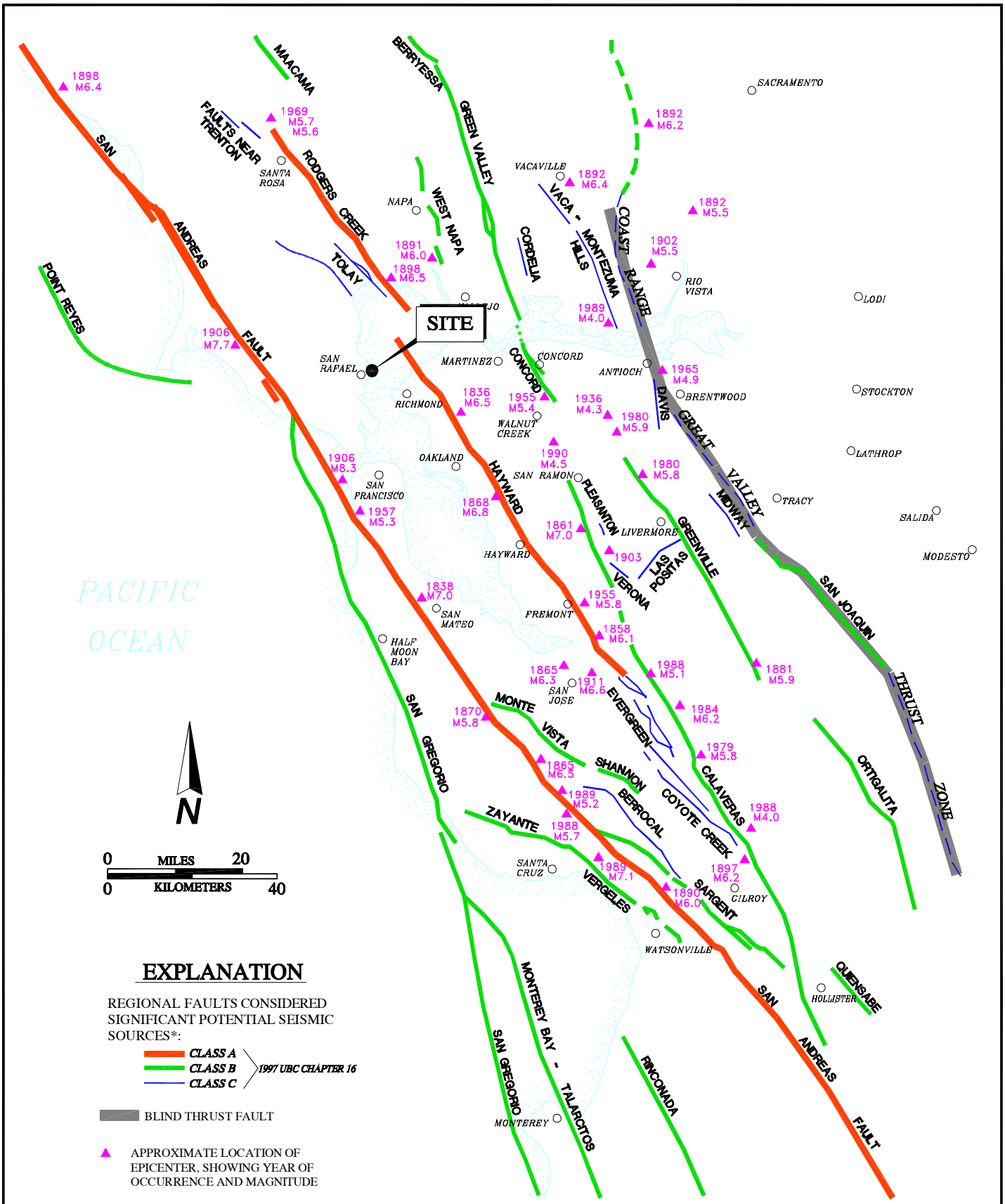
DRAWN BY: PC / CHECKED BY: PJS	



*BASED ON USGS OPEN FILE 96-706

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

REGIONAL FAULTING AND SEISMICITY

SAN RAFAEL QUARRY

SAN RAFAEL, CALIFORNIA

PROJECT NO.:	6261.1.003.01
DATE:	APRIL 2005
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FIGURE NO. 4

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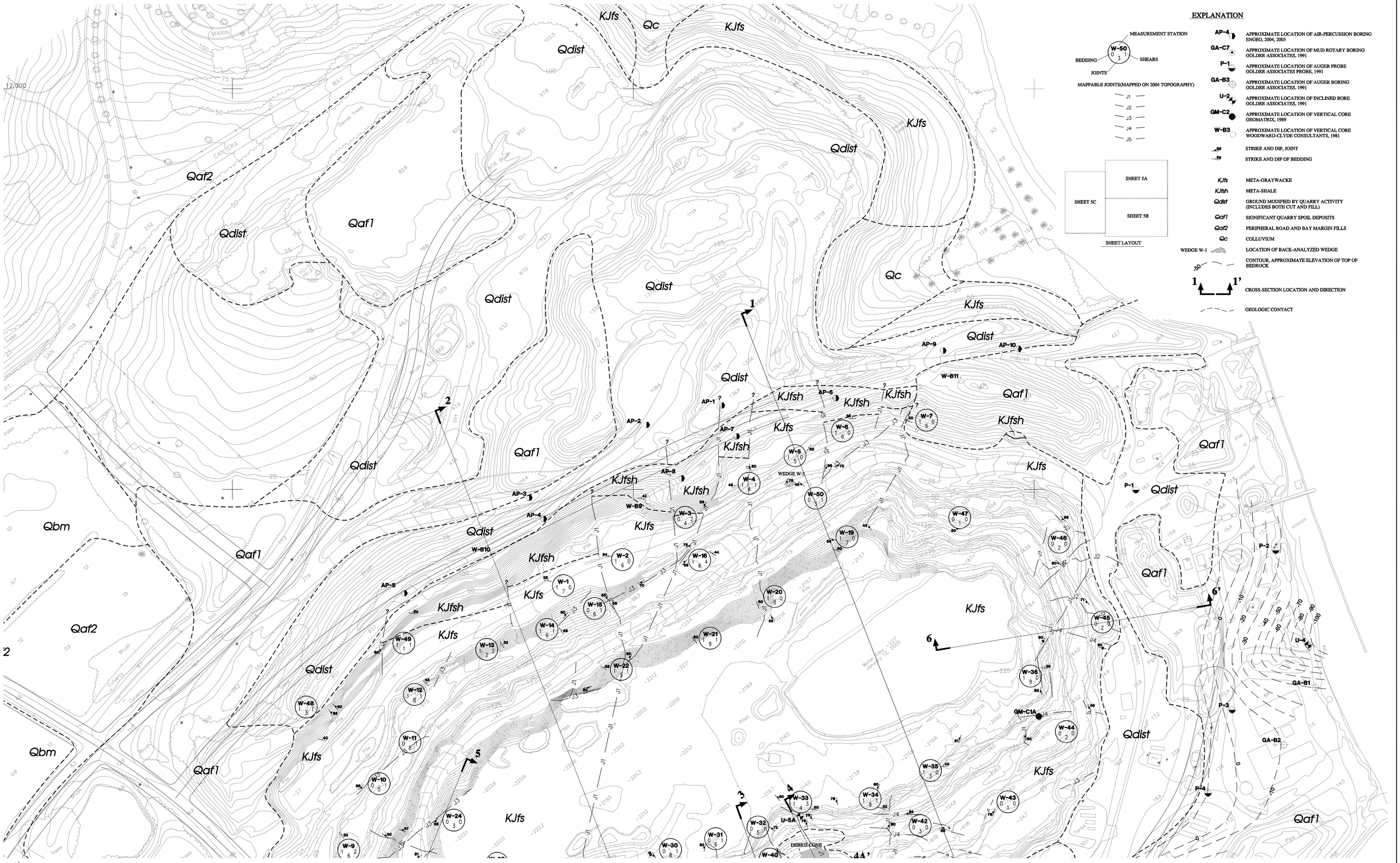

BASE MAP SOURCE:CSW STUBER/STROEH, 2005

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

SITE GEOLOGIC MAP
SAN RAFAEL QUARRY
MARIN COUNTY, CALIFORNIA

PROJECT NO.: 6261.1.003.01		FIGURE NO.
DATE: APRIL, 2005		5A
DRAWN BY: PC	CHECKED BY: PJS	

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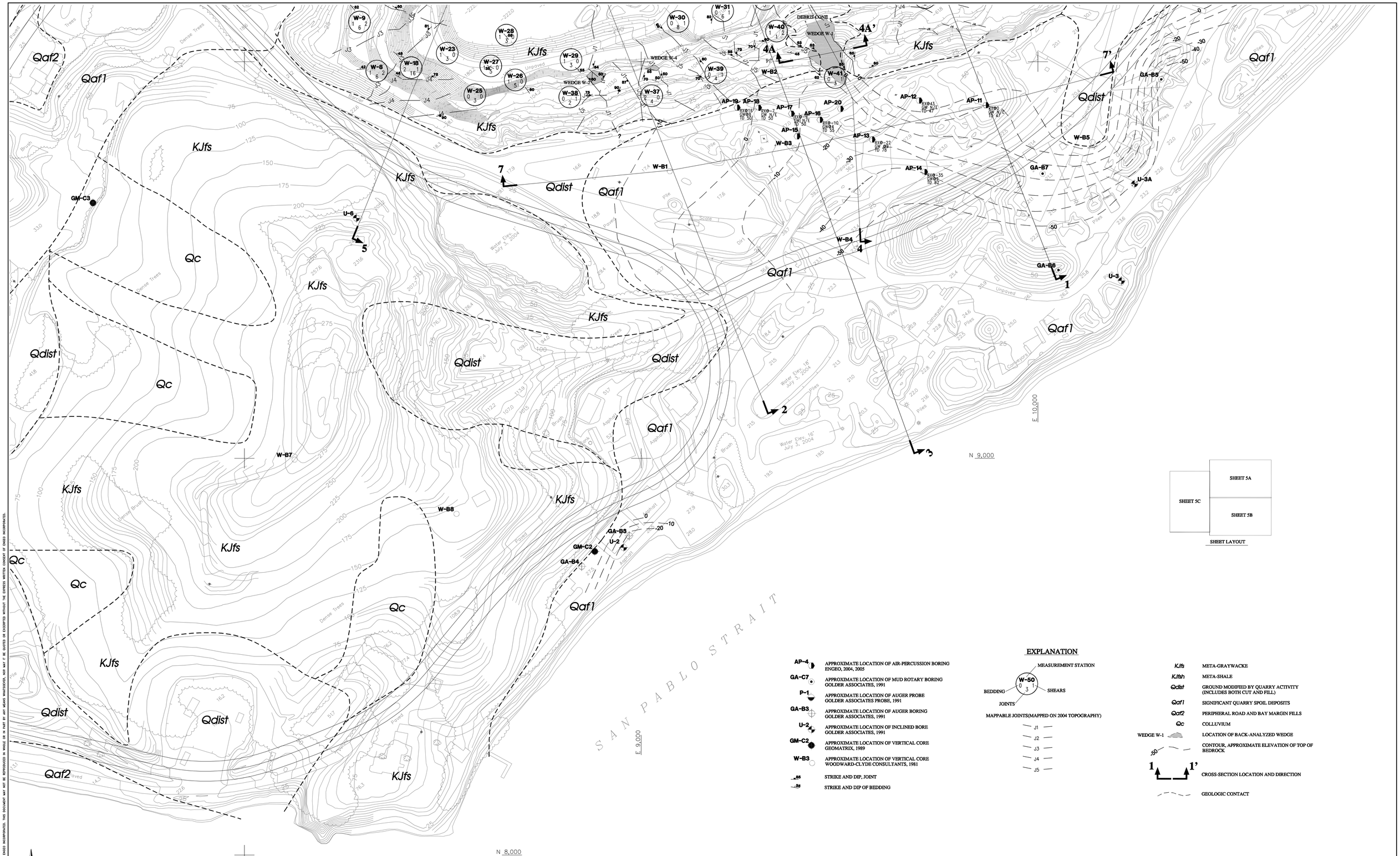

SHEET LAYOUT

EXPLANATION

AP-4	APPROXIMATE LOCATION OF AIR-PERCUSSION BORING ENGEO, 2004, 2005
GA-C7	APPROXIMATE LOCATION OF MUD ROTARY BORING GOLDER ASSOCIATES, 1991
P-1	APPROXIMATE LOCATION OF AUGER PROBE GOLDER ASSOCIATES PROBE, 1991
GA-B3	APPROXIMATE LOCATION OF AUGER BORING GOLDER ASSOCIATES, 1991
U-2	APPROXIMATE LOCATION OF INCLINED BORE GOLDER ASSOCIATES, 1991
GM-C2	APPROXIMATE LOCATION OF VERTICAL CORE GEOMATRIX, 1989
W-B3	APPROXIMATE LOCATION OF VERTICAL CORE WOODWARD-CLYDE CONSULTANTS, 1981
85	STRIKE AND DIP, JOINT
35	STRIKE AND DIP OF BEDDING

W-50 — MEASUREMENT STATION; 0 1 3 — SHEARS; BEDDING; JOINTS

MAPPABLE JOINTS(MAPPED ON 2004 TOPOGRAPHY)

J1
J2
J3
J4
J5

KJfs	META-GRAYWACKE
KJfsh	META-SHALE
Qdist	GROUND MODIFIED BY QUARRY ACTIVITY (INCLUDES BOTH CUT AND FILL)
Qaf1	SIGNIFICANT QUARRY SPOIL DEPOSITS
Qaf2	PERIPHERAL ROAD AND BAY MARGIN FILLS
Qc	COLLUVIUM
WEDGE W-1	LOCATION OF BACK-ANALYZED WEDGE
50	CONTOUR, APPROXIMATE ELEVATION OF TOP OF BEDROCK
1 — 1'	CROSS-SECTION LOCATION AND DIRECTION
	GEOLOGIC CONTACT

BASE MAP SOURCE:CSW STUBER/STROEH, 2005

ENGEO INCORPORATED — EXCELLENT SERVICE SINCE 1971

SITE GEOLOGIC MAP
SAN RAFAEL QUARRY
MARIN COUNTY, CALIFORNIA

PROJECT NO.: 6261.1.003.01
DATE: APRIL, 2005
DRAWN BY: PC
CHECKED BY: PJS
FIGURE NO. 5B

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EXPLANATION

MEASUREMENT STATION
W-50
0 3 1
BEDDING
SHEARS
JOINTS

MAPPABLE JOINTS(MAPPED ON 2004 TOPOGRAPHY)

J1
J2
J3
J4
J5

SHEET 5A
SHEET 5C
SHEET 5B

SHEET LAYOUT

AP-4 APPROXIMATE LOCATION OF AIR-PERCUSSION BORING ENGEO, 2004, 2005

GA-C7 APPROXIMATE LOCATION OF MUD ROTARY BORING GOLDER ASSOCIATES, 1991

P-1 APPROXIMATE LOCATION OF AUGER PROBE GOLDER ASSOCIATES PROBE, 1991

GA-B3 APPROXIMATE LOCATION OF AUGER BORING GOLDER ASSOCIATES, 1991

U-2 APPROXIMATE LOCATION OF INCLINED BORE GOLDER ASSOCIATES, 1991

GM-C2 APPROXIMATE LOCATION OF VERTICAL CORE GEOMATRIX, 1989

W-B3 APPROXIMATE LOCATION OF VERTICAL CORE WOODWARD-CLYDE CONSULTANTS, 1981

85 STRIKE AND DIP, JOINT

35 STRIKE AND DIP OF BEDDING

KJfs META-GRAYWACKE

KJfsh META-SHALE

Qdist GROUND MODIFIED BY QUARRY ACTIVITY (INCLUDES BOTH CUT AND FILL)

Qaf1 SIGNIFICANT QUARRY SPOIL DEPOSITS

Qaf2 PERIPHERAL ROAD AND BAY MARGIN FILLS

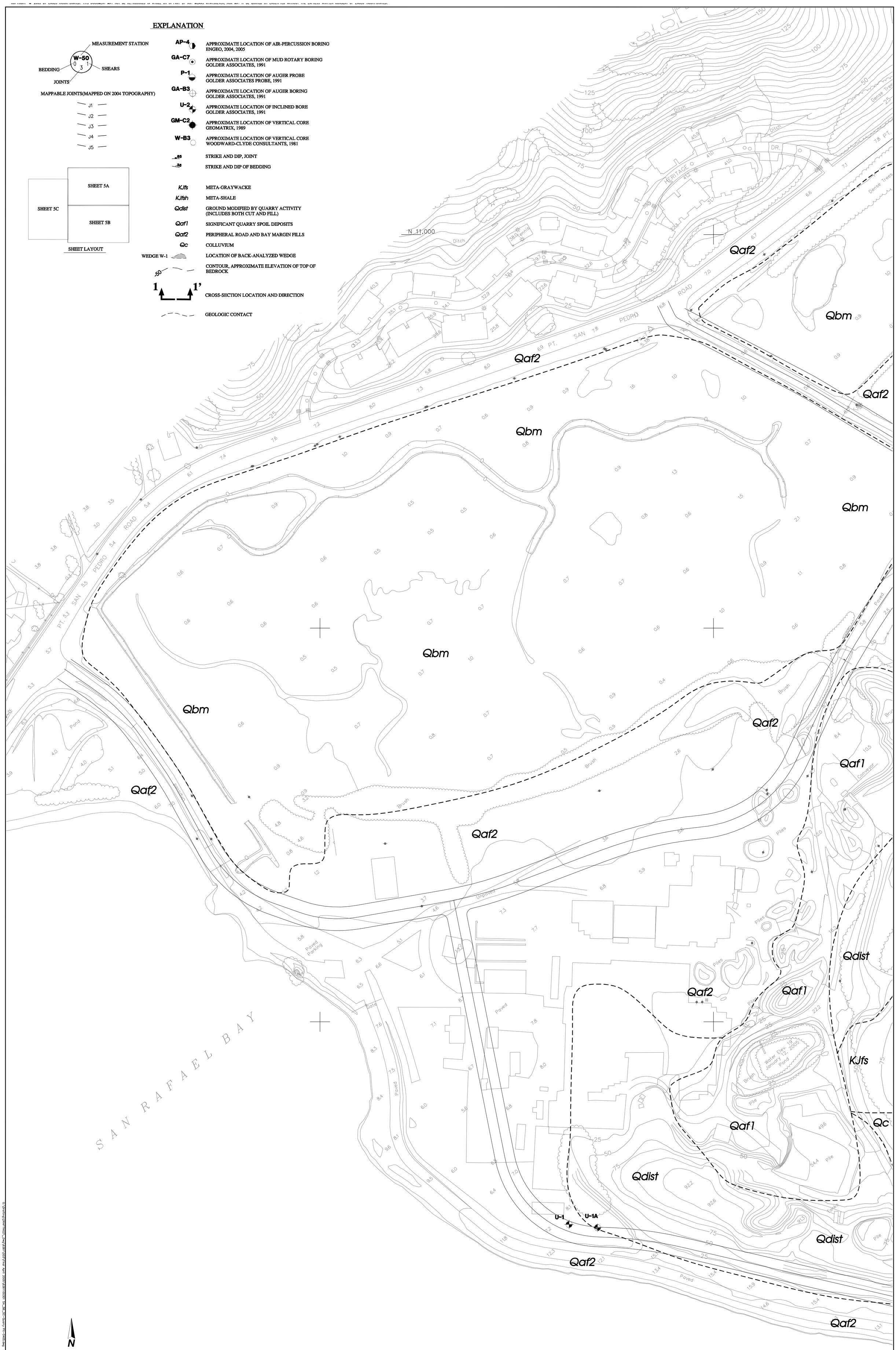
Qc COLLUVIUM

WEDGE W-1 LOCATION OF BACK-ANALYZED WEDGE

-50 CONTOUR, APPROXIMATE ELEVATION OF TOP OF BEDROCK

1 1' CROSS-SECTION LOCATION AND DIRECTION

GEOLOGIC CONTACT



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ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

SITE GEOLOGIC MAP
SAN RAFAEL QUARRY
MARIN COUNTY, CALIFORNIA

PROJECT NO.: 6261.1.003.01	FIGURE NO.
DATE: APRIL 2005	5C
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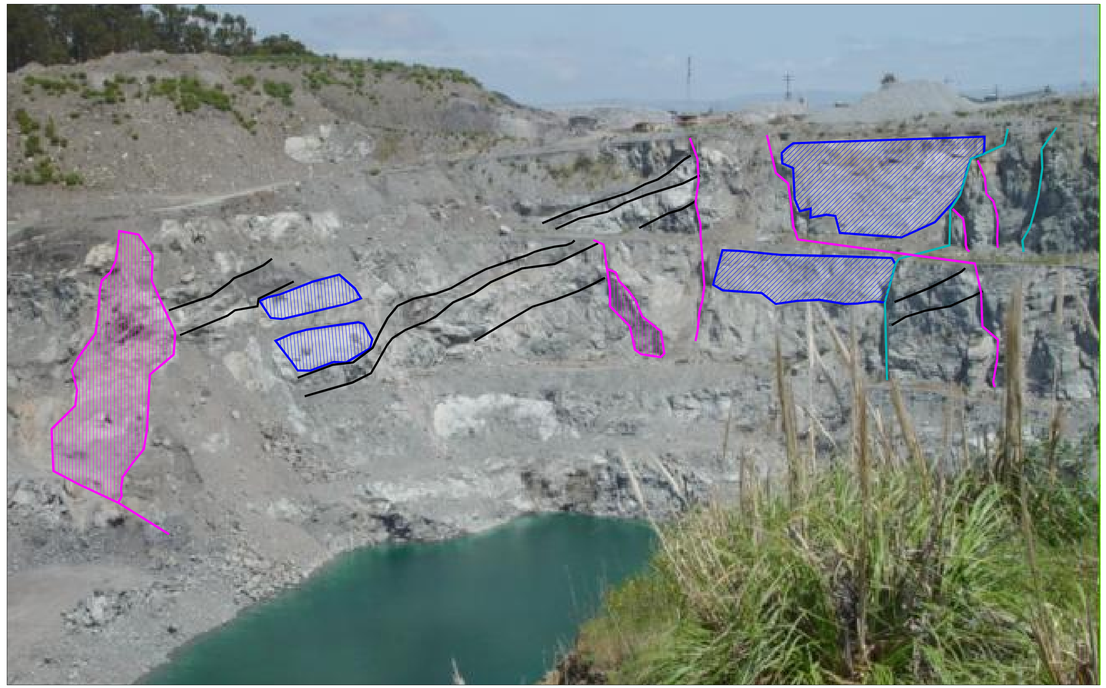
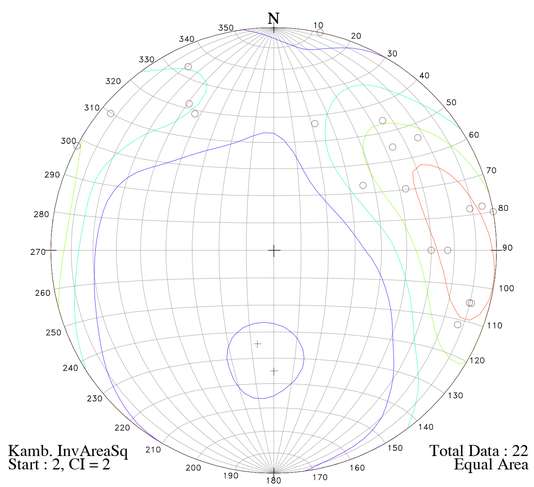


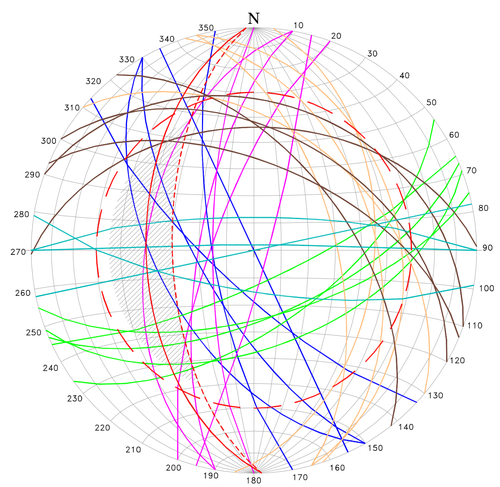
SITE PHOTOGRAPH (MAY 2004) - EAST FACE (N.T.S.)

EXPLANATION

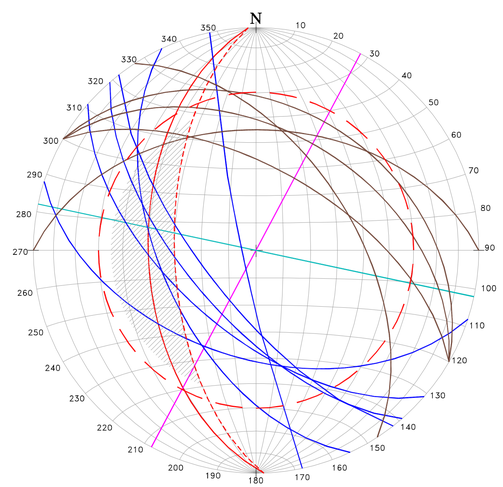
- APPROXIMATE LOCATION OF PROMINENT JOINT PLANE IN PHOTOGRAPH
- APPROXIMATE EXTENT OF PROMINENT JOINT FACE EXPOSURE (COLOR VARIES BASED ON JOINT SET)
- JOINT SET J1 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J2 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J3 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J4 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J5 ON PHOTOGRAPH AND STEREONET PLOT
- BEDDING PLANE ON PHOTOGRAPH AND STEREONET PLOT
- VISIBLY WEATHERED ROCK ZONE IN SOUTH FACE
- POLE OF BEDDING PLANE ON STEREONET PLOT
- POLE OF JOINT ON STEREONET PLOT
- POLE OF MAPPABLE JOINT ON STEREONET PLOT
- JOINT SET CLUSTER BOUNDARY ON STEREONET PLOT
- APPROXIMATED EXISTING FACE OREINTATION ON STEREONET PLOT
- PROPOSED FACE ORIENTATION ON STEREONET PLOT
- FRICTION CIRCLE ON STEREONET PLOT
- ZONE OF DAYLIGHTING WEDGE INTERSECTIONS ON STEREONET PLOT



EAST WALL ALL DATA



EAST COMPILED CLUSTERS

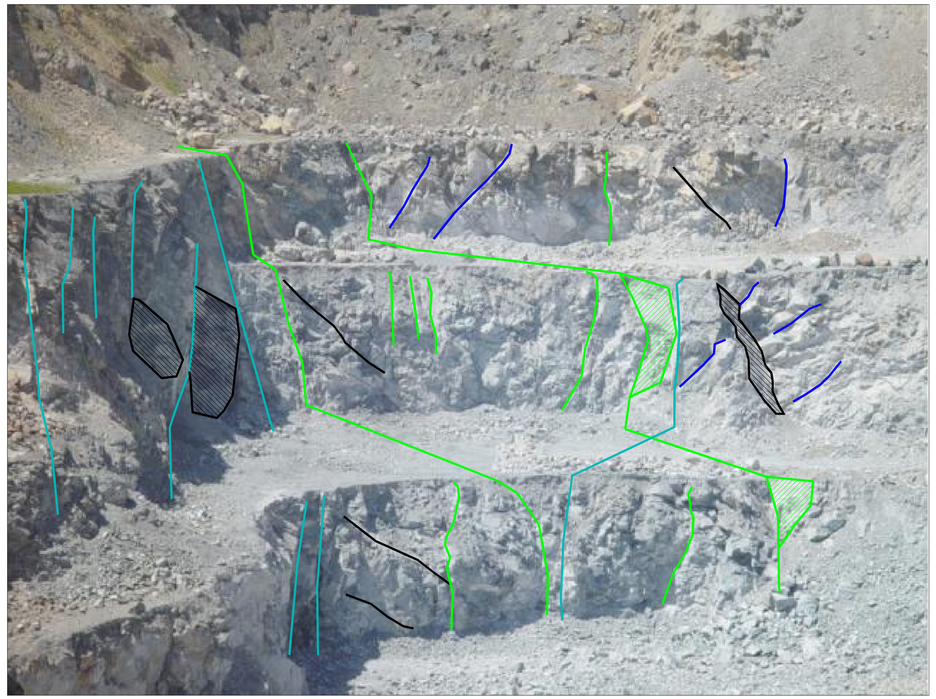


EAST WALL MAPPABLE
CONTINUOUS JOINTS PLUS BEDDING

NO SCALE

ENGEO INCORPORATED EXCELLENT WORK SINCE 1971	EAST FACE VIEW AND STEREONET PLOTS SAN RAFAEL QUARRY SAN RAFAEL, CALIFORNIA	PROJECT NO.: 6261.1.003.01 DATE: APRIL 2005 DRAWN BY: PC CHECKED BY: PJS	FIGURE NO. 6

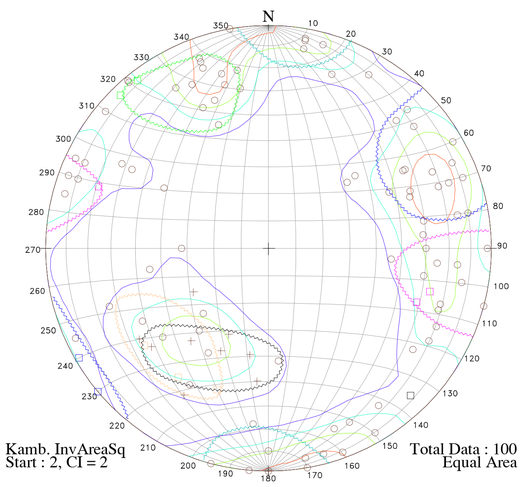
ORIGINAL FIGURE PRINTED IN COLOR



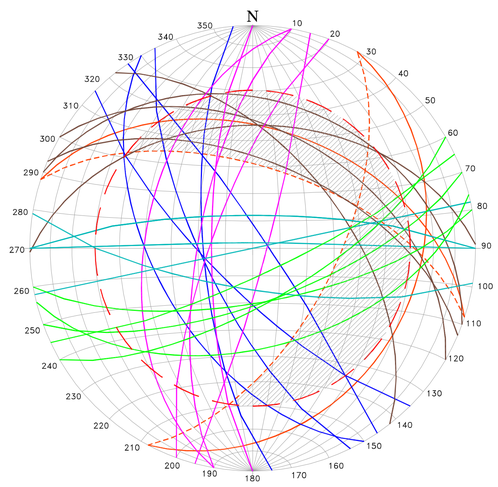
SITE PHOTOGRAPH (MAY 2004) - WEST FACE(N.T.S.)

EXPLANATION

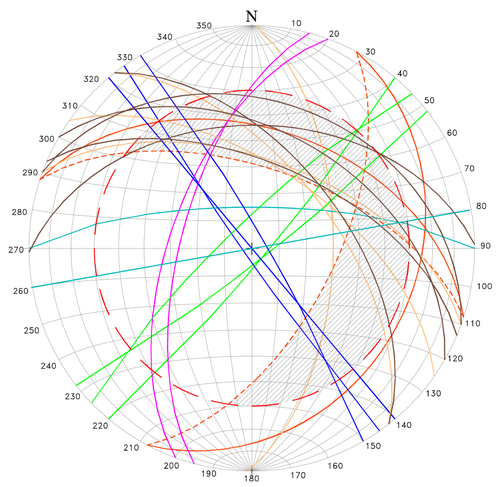
- APPROXIMATE LOCATION OF PROMINENT JOINT PLANE IN PHOTOGRAPH
- APPROXIMATE EXTENT OF PROMINENT JOINT FACE EXPOSURE (COLOR VARIES BASED ON JOINT SET)
- JOINT SET J1 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J2 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J3 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J4 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J5 ON PHOTOGRAPH AND STEREONET PLOT
- BEDDING PLANE ON PHOTOGRAPH AND STEREONET PLOT
- VISIBLY WEATHERED ROCK ZONE IN SOUTH FACE
- POLE OF BEDDING PLANE ON STEREONET PLOT
- POLE OF JOINT ON STEREONET PLOT
- POLE OF MAPPABLE JOINT ON STEREONET PLOT
- JOINT SET CLUSTER BOUNDARY ON STEREONET PLOT
- APPROXIMATED EXISTING FACE OREINTATION ON STEREONET PLOT
- PROPOSED FACE ORIENTATION ON STEREONET PLOT
- FRICTION CIRCLE ON STEREONET PLOT
- ZONE OF DAYLIGHTING WEDGE INTERSECTIONS ON STEREONET PLOT



WEST WALL ALL DATA



WEST COMPILED CLUSTERS



WEST WALL MAPPABLE
CONTINUOUS JOINTS PLUS BEDDING

NO SCALE

ENGEO INCORPORATED EXCELLENT SERVICE SINCE 1971	WEST FACE VIEW AND STEREONET PLOTS SAN RAFAEL QUARRY SAN RAFAEL, CALIFORNIA	PROJECT NO.: 6261.1.003.01 DATE: APRIL 2005 DRAWN BY: PC CHECKED BY: PJS	FIGURE NO. 7

ORIGINAL FIGURE PRINTED IN COLOR

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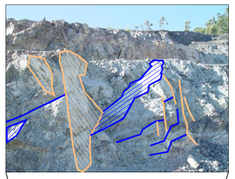
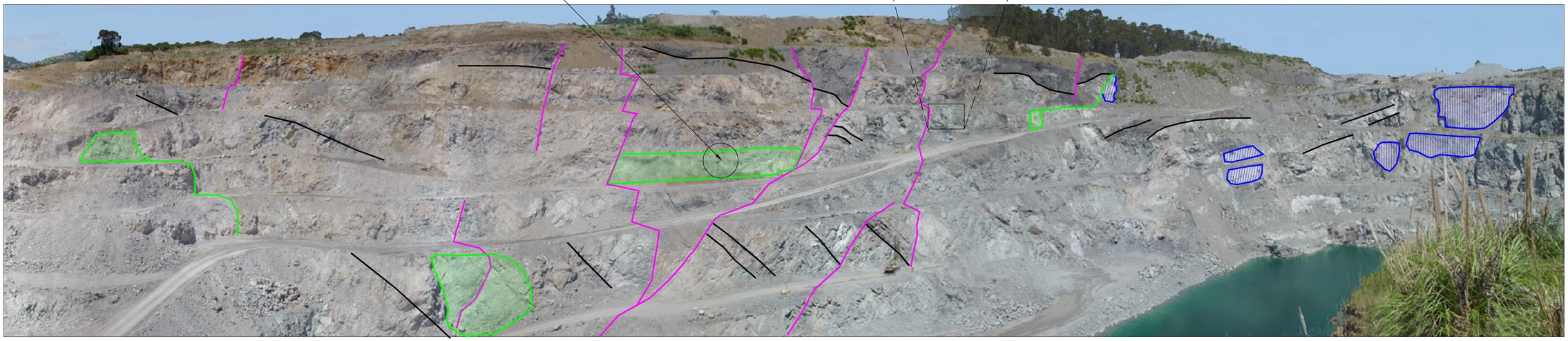
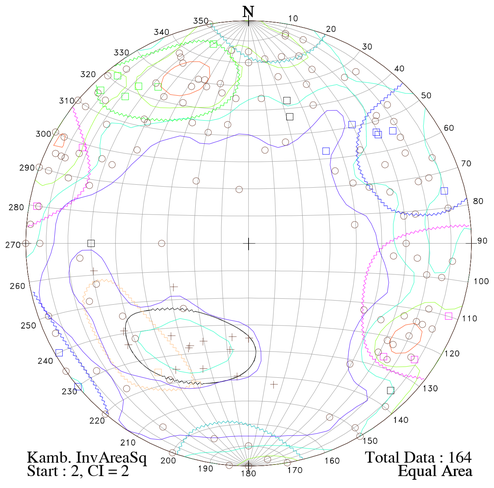
EXPOSED JOINT SURFACE
INTER-BENCH FACE
SET J3



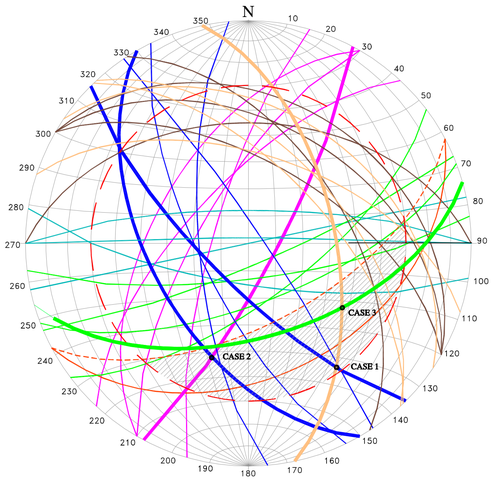
WEDGE W-3



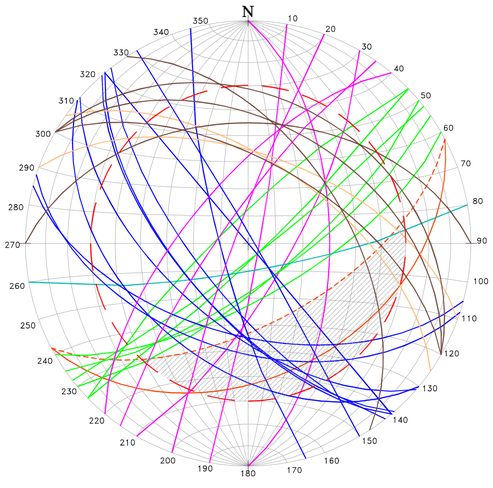
SITE PHOTOGRAPH (MAY 2004) - NORTH FACE (N.T.S.)



NORTH WALL ALL DATA



NORTH COMPILED CLUSTERS



NORTH WALL MAPPABLE
CONTINUOUS JOINTS PLUS BEDDING

EXPLANATION

- APPROXIMATE LOCATION OF PROMINENT JOINT PLANE IN PHOTOGRAPH
- APPROXIMATE EXTENT OF PROMINENT JOINT FACE EXPOSURE (COLOR VARIES BASED ON JOINT SET)
- JOINT SET J1 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J2 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J3 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J4 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J5 ON PHOTOGRAPH AND STEREONET PLOT
- BEDDING PLANE ON PHOTOGRAPH AND STEREONET PLOT
- POLE OF BEDDING PLANE ON STEREONET PLOT
- POLE OF JOINT ON STEREONET PLOT
- POLE OF MAPPABLE JOINT ON STEREONET PLOT
- JOINT SET CLUSTER BOUNDARY ON STEREONET PLOT
- APPROXIMATED EXISTING FACE OREINTATION ON STEREONET PLOT
- PROPOSED FACE ORIENTATION ON STEREONET PLOT
- FRICTION CIRCLE ON STEREONET PLOT
- ZONE OF DAYLIGHTING WEDGE INTERSECTIONS ON STEREONET PLOT

NO SCALE

ENGEO INCORPORATED EXCELLENT SERVICE SINCE 1971	NORTH FACE VIEW AND STEREONET PLOTS SAN RAFAEL QUARRY SAN RAFAEL, CALIFORNIA	PROJECT NO.: 6261.1.003.01 DATE: APRIL 2005 DRAWN BY: PC CHECKED BY: PJS	FIGURE NO. 8

ORIGINAL FIGURE PRINTED IN COLOR

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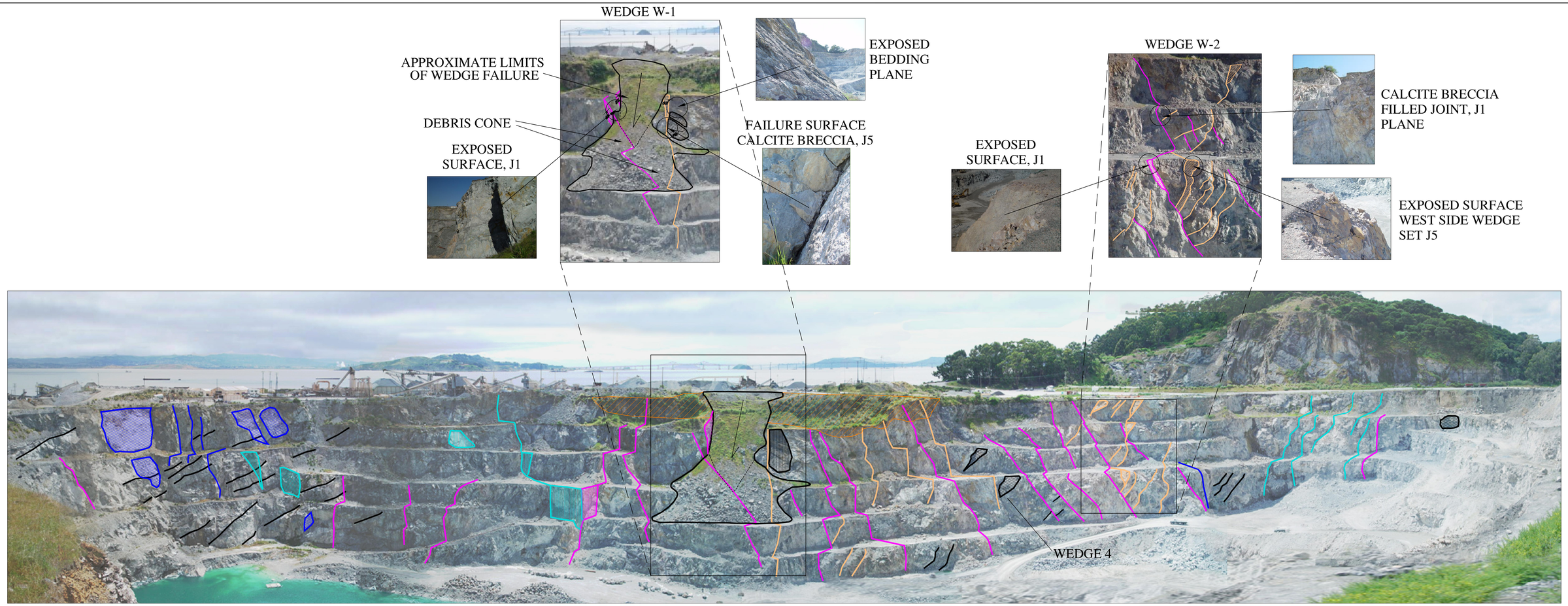
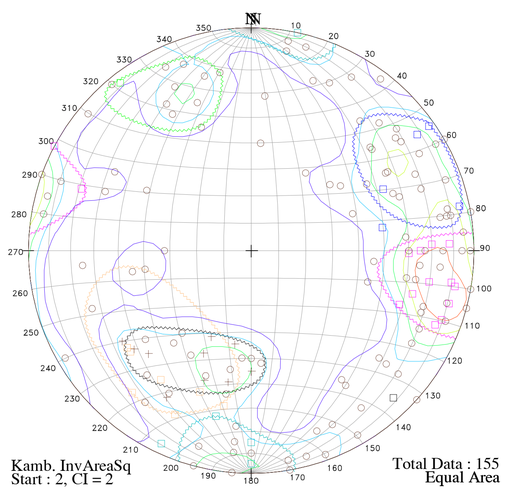
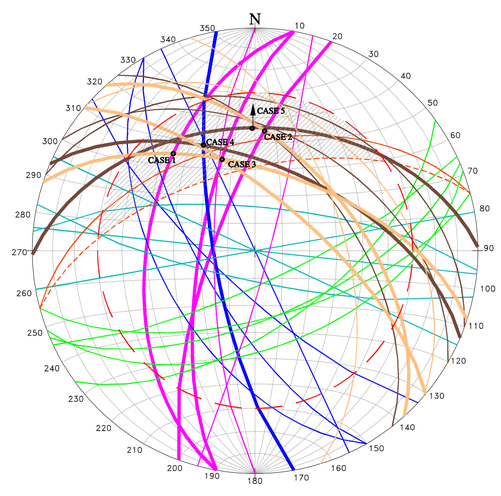


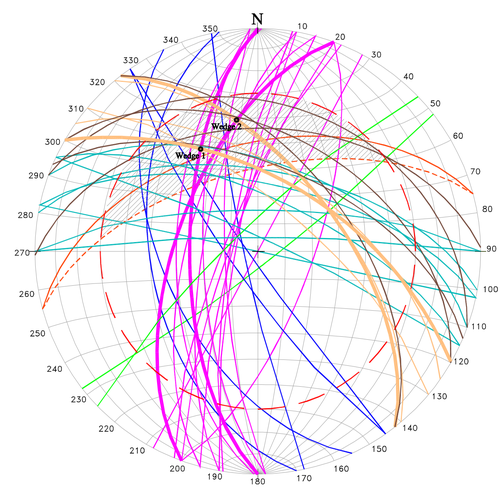
SITE PHOTOGRAPH (MAY 2004) - SOUTH FACE (N.T.S)



SOUTH WALL ALL DATA



SOUTH COMPILED CLUSTERS



SOUTH WALL MAPPABLE CONTINUOUS JOINTS PLUS BEDDING

EXPLANATION

- APPROXIMATE LOCATION OF PROMINENT JOINT PLANE IN PHOTOGRAPH
- APPROXIMATE EXTENT OF PROMINENT JOINT FACE EXPOSURE (COLOR VARIES BASED ON JOINT SET)
- JOINT SET J1 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J2 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J3 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J4 ON PHOTOGRAPH AND STEREONET PLOT
- JOINT SET J5 ON PHOTOGRAPH AND STEREONET PLOT
- BEDDING PLANE ON PHOTOGRAPH AND STEREONET PLOT
- VISIBLY WEATHERED ROCK ZONE IN SOUTH FACE
- POLE OF BEDDING PLANE ON STEREONET PLOT
- POLE OF JOINT ON STEREONET PLOT
- POLE OF MAPPABLE JOINT ON STEREONET PLOT
- JOINT SET CLUSTER BOUNDARY ON STEREONET PLOT
- APPROXIMATED EXISTING FACE OREINTATION ON STEREONET PLOT
- PROPOSED FACE ORIENTATION ON STEREONET PLOT
- FRICTION CIRCLE ON STEREONET PLOT
- ZONE OF DAYLIGHTING WEDGE INTERSECTIONS ON STEREONET PLOT

NO SCALE

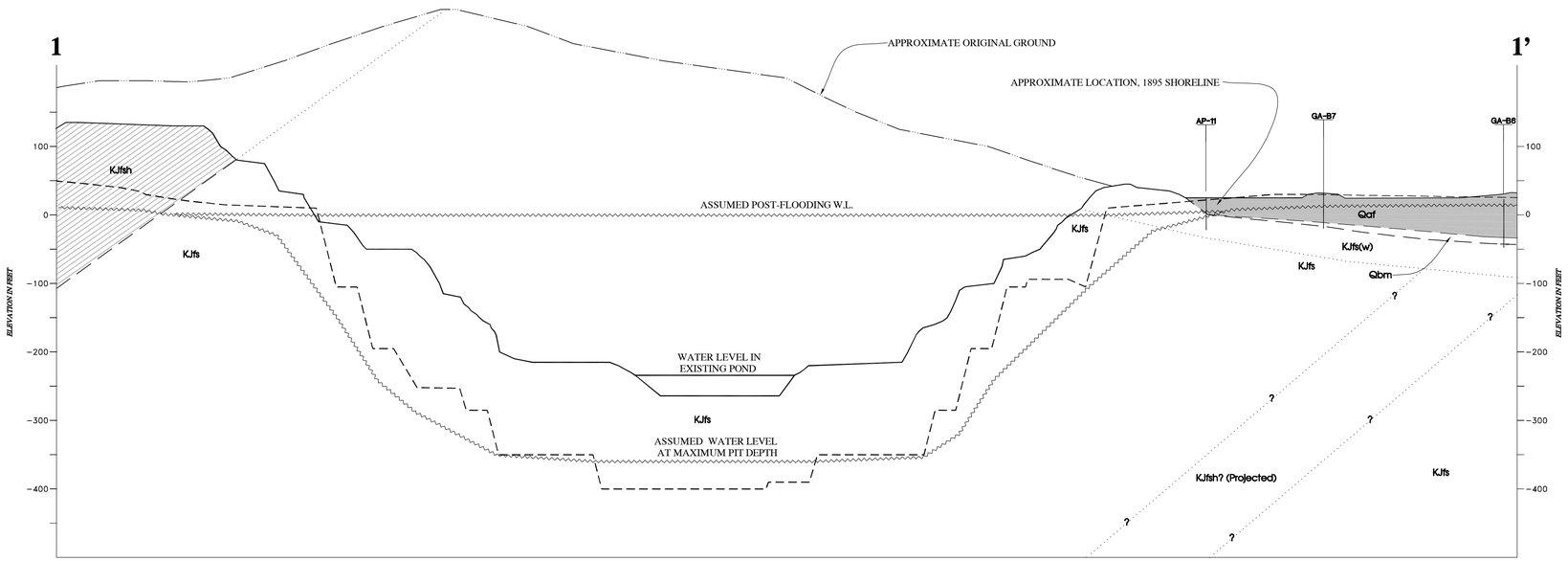
ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

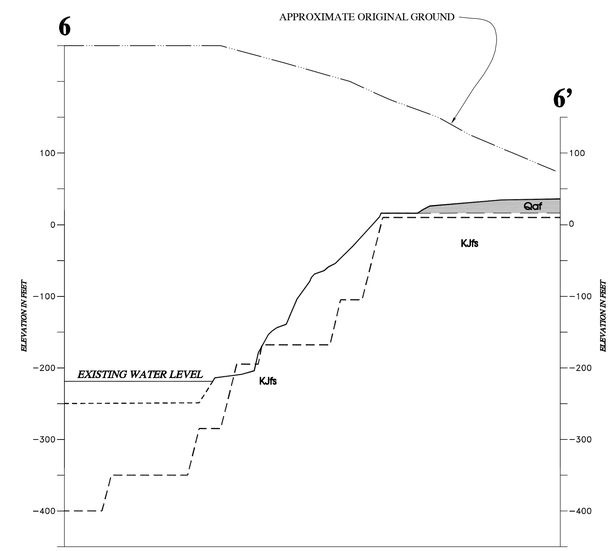
SOUTH FACE VIEW AND STEREONET PLOTS
SAN RAFAEL QUARRY
SAN RAFAEL, CALIFORNIA

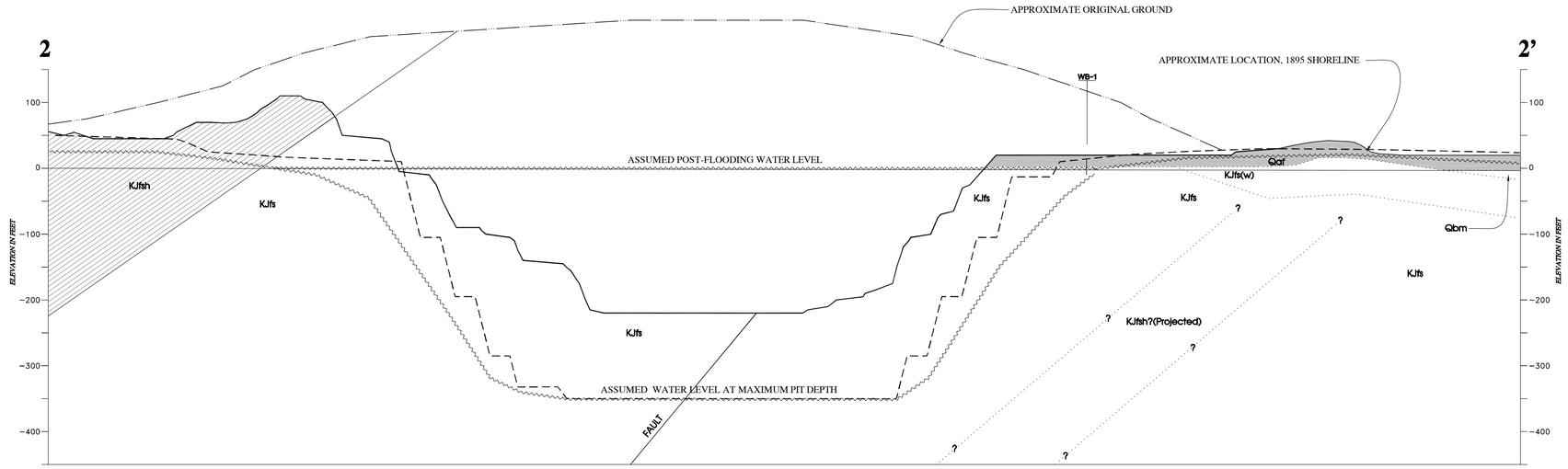
PROJECT NO.: 6261.1.003.01
DATE: APRIL 2005
DRAWN BY: PC
CHECKED BY: PJS

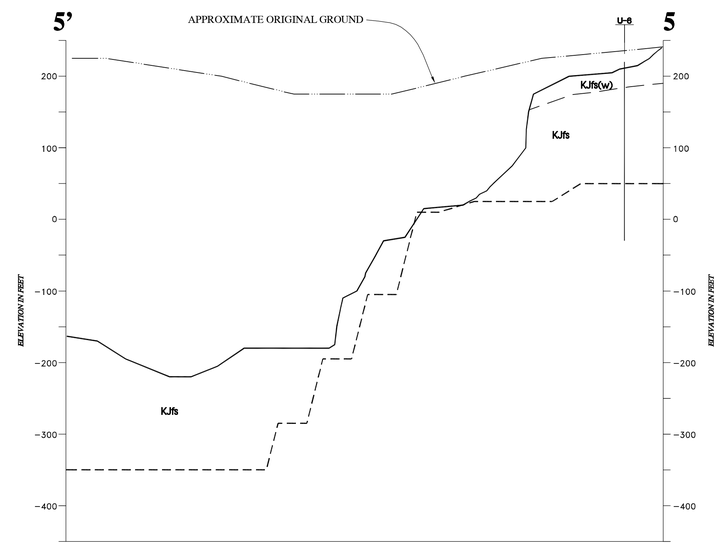
FIGURE NO. 9

ORIGINAL FIGURE PRINTED IN COLOR









EXPLANATION

Qaf	QUARRY FILLS
Qc/Qal	COLLUVIUM / ALLUVIUM
Qs	SAND AND GRAVEL (BEACH DEPOSITES)
Qbm	SILTY TO SANDY CLAY
Kjfs(w)	WEATHERED FRANCISCAN BEDROCK
KJfs	META GREYWACKE, FRANCISCAN COMPLEX
KJfsh	METASHALE, FRANCISCAN COMPLEX

U-8 — BORING LOCATION ON CROSS-SECTION

WATER LEVELS ASSUMED PER ANALYSIS

EXISTING GROUND

PROPOSED GRADE

APPROXIMATE ORIGINAL GROUND SURFACE (1895)

GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE, DOTTED WHERE INFERRED

1" = 100'

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

CROSS SECTIONS
SAN RAFAEL QUARRY
SAN RAFAEL, CALIFORNIA

PROJECT NO.: 6261.1.003.03			FIGURE NO.
DATE: APRIL 2005			10
DRAWN BY: PC	CHECKED BY: PJS		

ORIGINAL FIGURE PRINTED IN COLOR

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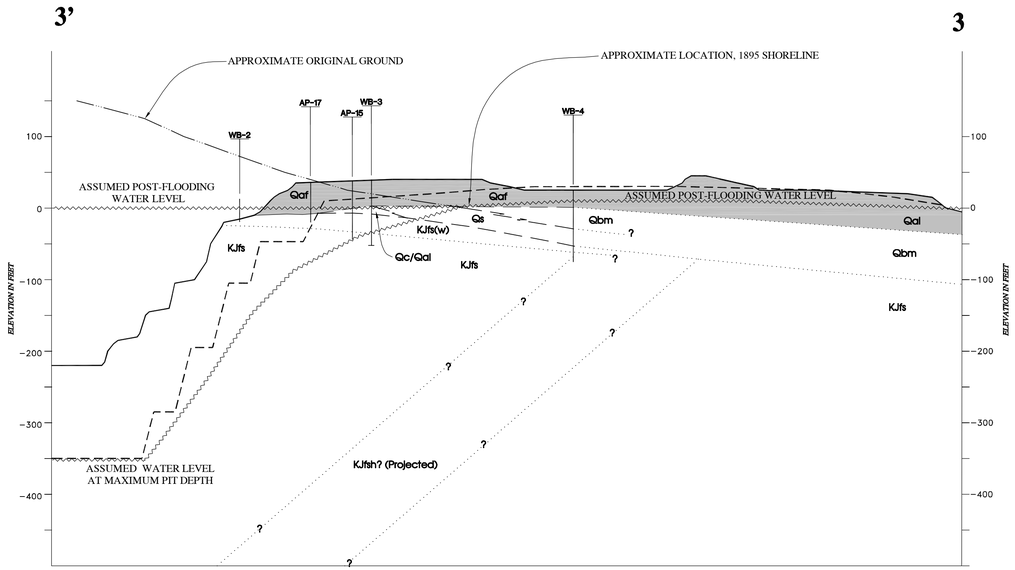

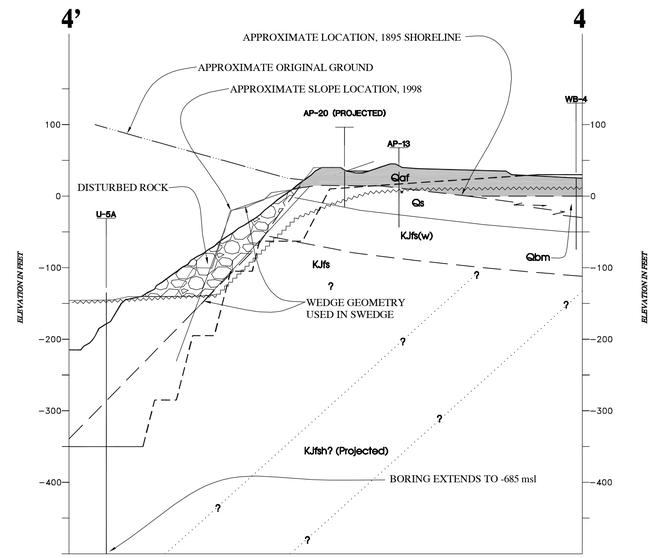

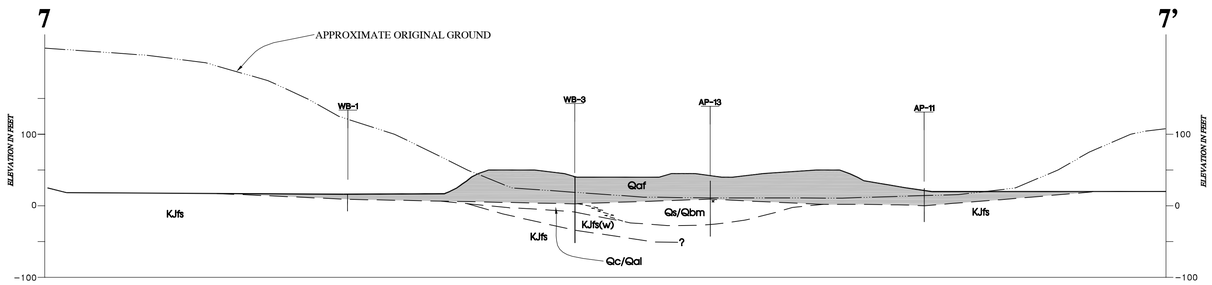

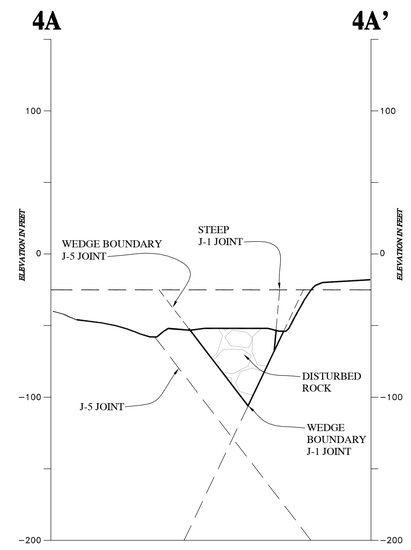

EXPLANATION

Qaf	QUARRY FILLS
Qc/Qal	COLLUVIUM / ALLUVIUM
Qs	SAND AND GRAVEL (BEACH DEPOSITES)
Qbm	SILTY TO SANDY CLAY
KJfs(w)	WEATHERED FRANCISCAN BEDROCK
KJfs	META GREYWACKE, FRANCISCAN COMPLEX
KJfsh	METASHALE, FRANCISCAN COMPLEX

U-6 — BORING LOCATION ON CROSS-SECTION

WATER LEVELS ASSUMED PER ANALYSIS

EXISTING GROUND

PROPOSED GRADE

APPROXIMATE ORIGINAL GROUND SURFACE (1895)

GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE, DOTTED WHERE INFERRED

1" = 100'

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

CROSS SECTIONS
SAN RAFAEL QUARRY
SAN RAFAEL, CALIFORNIA

PROJECT NO.: 6261.1.003.03			FIGURE NO.
DATE: APRIL 2005			11
DRAWN BY: PC	CHECKED BY: PJS		

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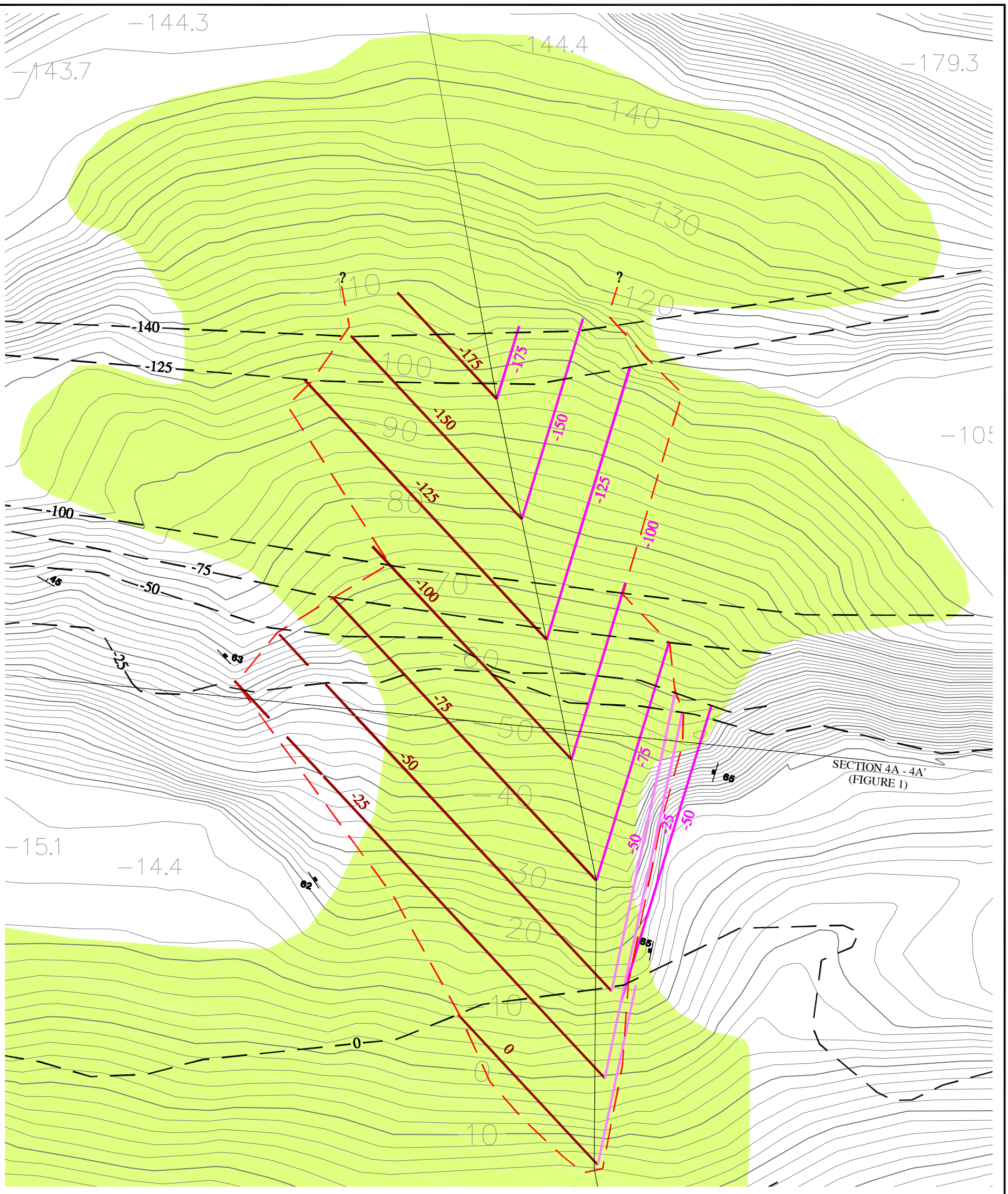
ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

- APPROXIMATE LIMIT OF WEDGE FAILURE (PROJECTS TO 1998 SURFACE)
- CONTOUR ON JOINT J-1 (65° DIP)
- CONTOUR ON JOINT J-1 (85° DIP)
- CONTOUR ON JOINT J-5
- APPROXIMATE BENCH CONTOUR AT TIME OF FAILURE (ESTIMATED FROM 1998 TOPOGRAPHY AND VERBAL ACCOUNTS)
- 45 STRIKE AND DIP OF JOINT
- 63 STRIKE AND DIP OF BEDDING
- DEBRIS CONE/FILL

N

0 FEET 20
0 METERS 10

BASE MAP SOURCE: TOWILL, INC.

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

WEDGE W-1
SAN RAFAEL QUARRY
MARIN COUNTY, CALIFORNIA

PROJECT NO.:	6261.1.003.01
DATE:	APRIL, 2005
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FIGURE NO. 12

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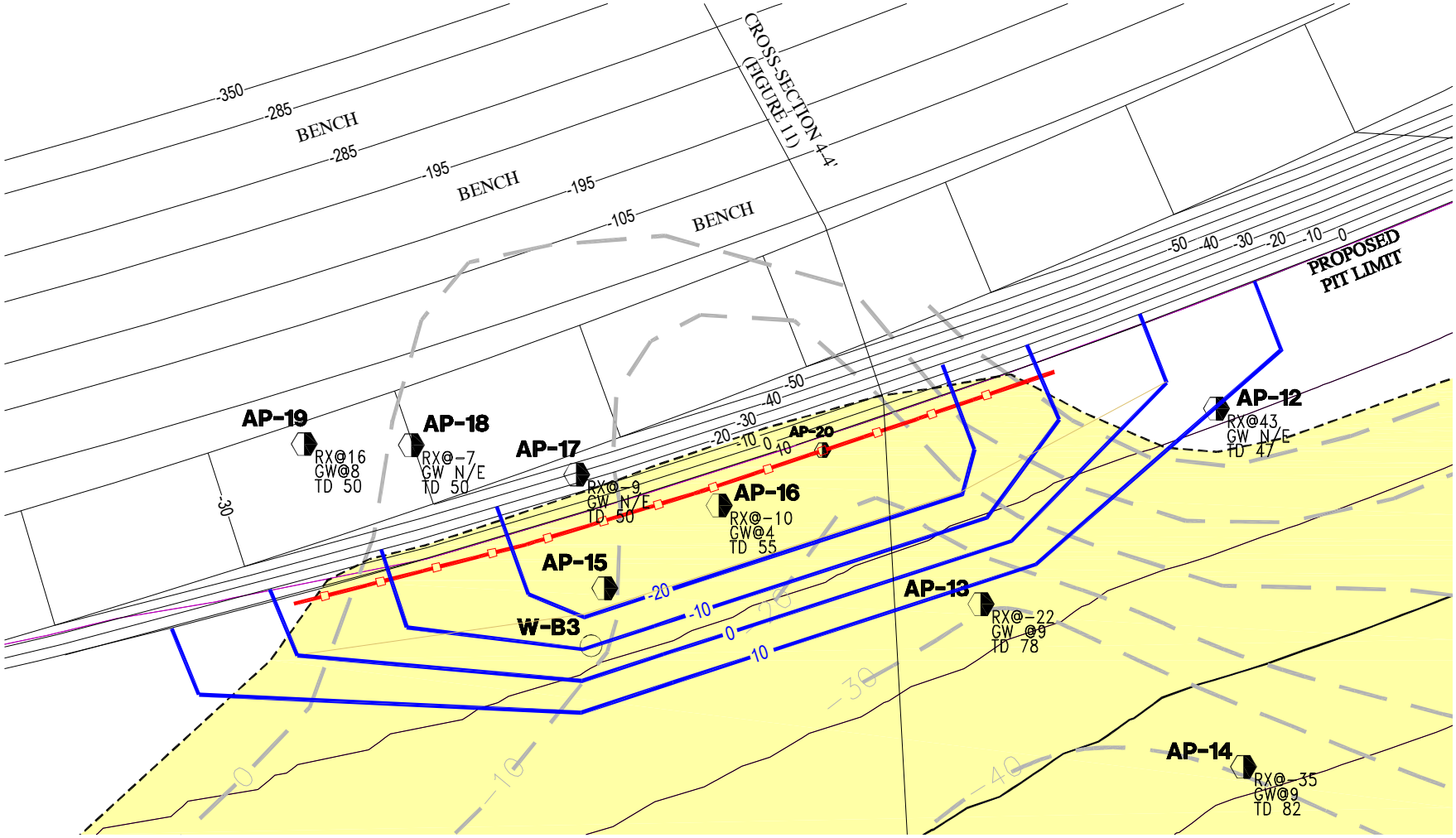


EXPLANATION

- ESTIMATED CONTOUR, TOP OF BED ROCK
- PROPOSED BUTTRESS KEY EXCAVATION
- PROPOSED SHEET PILE WALL LOCATION
- ESTIMATED LIMITS OF SOIL DEPOSITS AFTER ARP04 GRADING

N

0 FEET 80
0 METERS 40

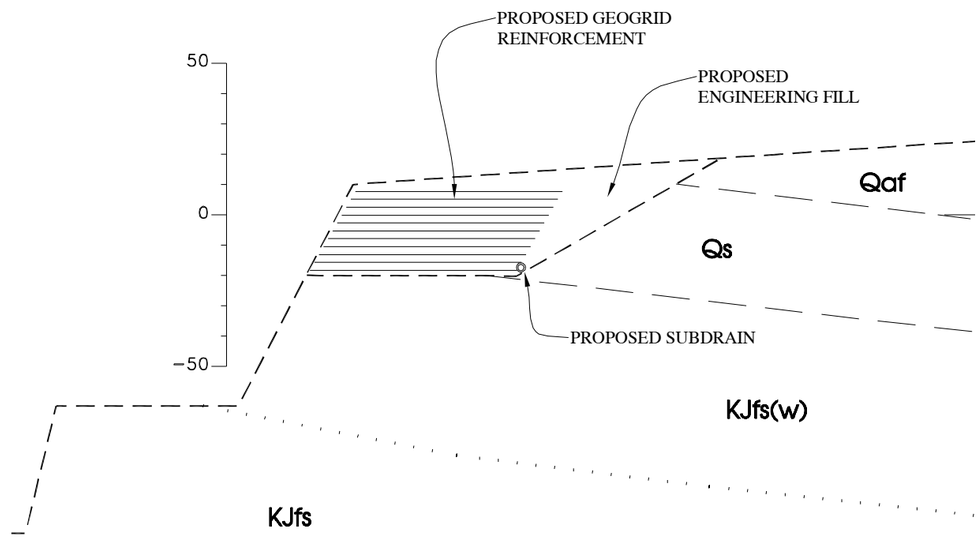
ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

GEOTECHNICAL OPTIONS, SOUTH BROW

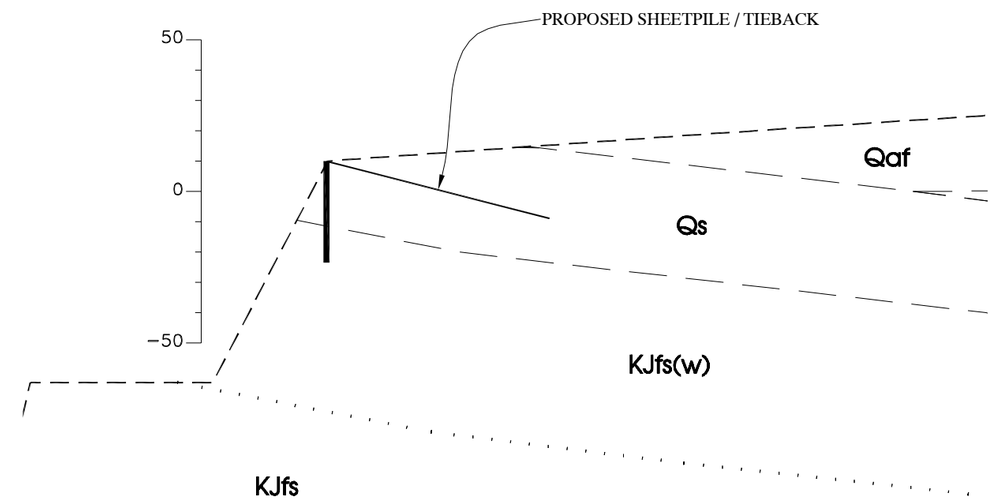
SAN RAFAEL QUARRY

MARIN COUNTY, CALIFORNIA

PROJECT NO.: 6261.1.003.01	FIGURE NO. 13
DATE: APRIL 2005	
DRAWN BY: PC	CHECKED BY: PJS



DETAIL OF SECTION 4-4'



DETAIL OF SECTION 4-4'

1" = 50'

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

CONCEPTUAL GEOTECHNICAL REMEDIATION DETAILS
SAN RAFAEL QUARRY
MARIN COUNTY, CALIFORNIA

PROJECT NO.: 6261.1.003.01
DATE: APRIL, 2005
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FIGURE NO. 14

ORIGINAL FIGURE PRINTED IN COLOR

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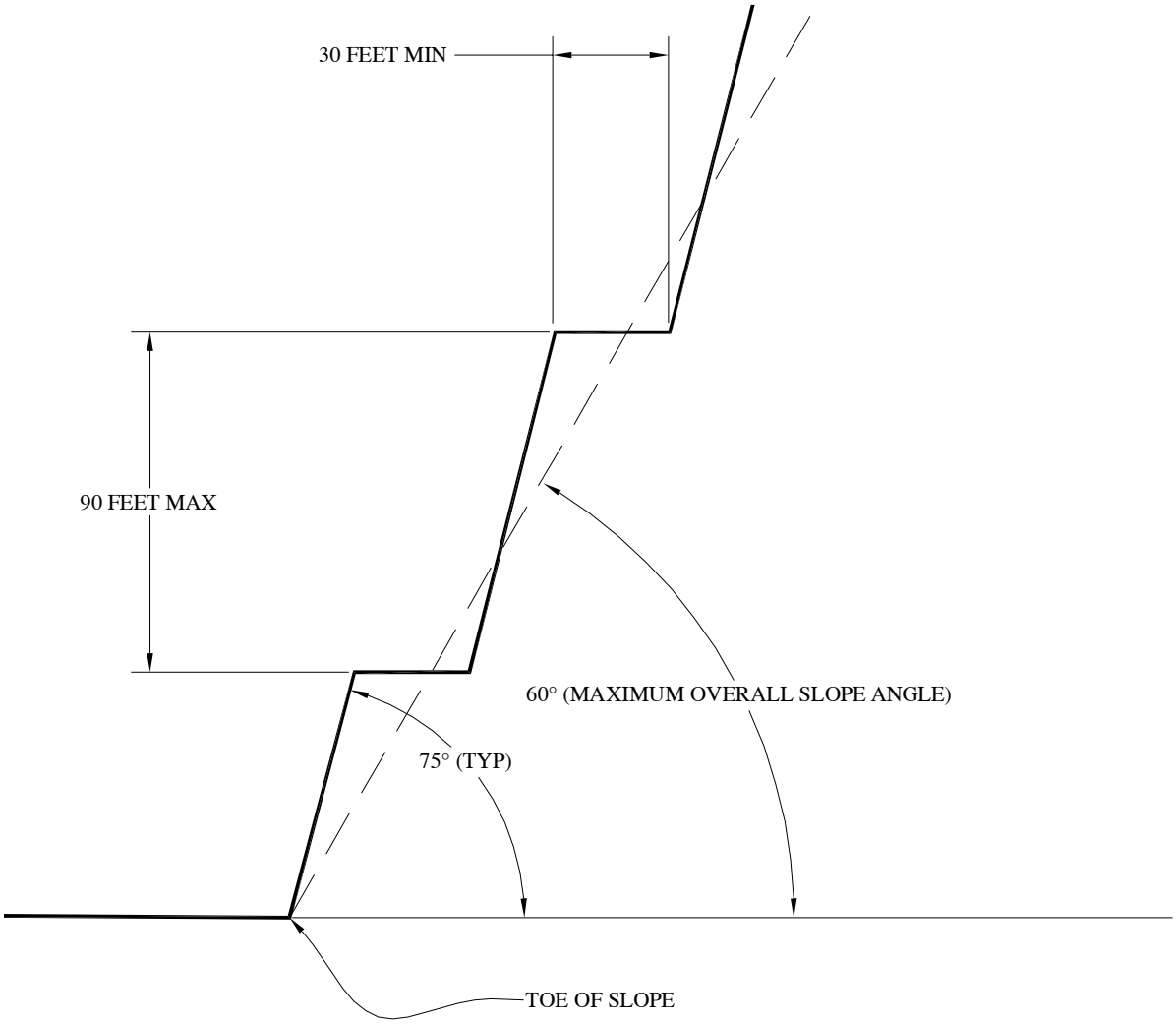

NO SCALE

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

RECOMMENDED SAFETY BENCH DETAIL
SAN RAFAEL QUARRY
MARIN COUNTY, CALIFORNIA

PROJECT NO.: 6261.1.003.01	FIGURE NO. 15
DATE: APRIL 2005	
DRAWN BY: PC	CHECKED BY: PJS

ORIGINAL FIGURE PRINTED IN COLOR

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APPENDIX A

Joint Characterization Data

JOINT DATA

Type	Strike	Dip (RHR)	Station	Roughness (JRC)	Surface Coating
BEDDING	280	35	1	5	NO FILL
JOINT	200	75	1	20	THIN CALCITE COATING
JOINT	190	70	1	15	THIN CALCITE COATING
JOINT	150	70	1	20	THIN CALCITE COATING
JOINT	120	75	1	20	THIN CALCITE COATING
JOINT	60	70	1	15	THIN CALCITE COATING
JOINT	0	90	1	15	THIN CALCITE COATING
BEDDING	260	54	2	5	NO FILL
JOINT	75	62	2	15	THIN CALCITE COATING
JOINT	0	83	2	15	THIN CALCITE COATING
JOINT	320	90	2	10	CALCITE VEIN
JOINT	265	87	2	20	NO FILL
JOINT	40	55	2	5	CALCITE VEIN
JOINT	5	88	2	15	THIN CALCITE COATING
JOINT	200	85	3	5	CALCITE/ROCK BRECCIA
JOINT	65	50	3	10	THIN CALCITE COATING
JOINT	340	85	3	15	THIN CALCITE COATING
JOINT	185	81	3	15	THIN CALCITE COATING
SHEAR	20	90	3	5	CALCITE/ROCK BRECCIA
SHEAR	0	90	3	5	CALCITE/ROCK BRECCIA
JOINT	240	90	4	10	NO FILL
JOINT	25	82	4	15	NO FILL
JOINT	28	75	4	20	NO FILL
JOINT	70	75	4	20	NO FILL
JOINT	90	75	4	20	NO FILL
SHEAR	165	80	4	5	CALCITE/ROCK BRECCIA
JOINT	215	65	5	20	THIN CALCITE COATING
JOINT	260	90	5	10	THIN CALCITE COATING
JOINT	30	85	5	15	THIN CALCITE COATING
JOINT	122	80	5	15	THIN CALCITE COATING
JOINT	280	78	5	20	THIN CALCITE COATING
JOINT	50	62	5	15	THIN CALCITE COATING
BEDDING	270	35	6	10	NO FILL
JOINT	355	65	6	15	THIN CALCITE COATING
JOINT	20	75	6	20	THIN CALCITE COATING
JOINT	85	55	6	20	THIN CALCITE COATING
JOINT	100	65	6	20	THIN CALCITE COATING
JOINT	50	30	6	5	THIN CALCITE COATING
JOINT	100	35	6	10	THIN CALCITE COATING
BEDDING	295	40	7	5	NO FILL
JOINT	136	68	7	10	THIN CALCITE COATING
JOINT	310	74	7	5	THIN CALCITE COATING
JOINT	55	85	7	15	NO FILL
JOINT	73	66	7	5	THIN CALCITE COATING
JOINT	70	44	7	5	THIN CALCITE COATING
JOINT	320	78	7	10	THIN CALCITE COATING
JOINT	68	50	8	5	NO FILL

Type	Strike	Dip (RHR)	Station	Roughness (JRC)	Surface Coating
JOINT	265	42	8	5	THIN CALCITE COATING
JOINT	275	68	8	15	THIN CALCITE COATING
JOINT	180	88	8	20	NO FILL
JOINT	160	70	8	15	THIN CALCITE COATING
JOINT	260	85	8	15	NO FILL
JOINT	250	75	8	15	THIN CALCITE COATING
SHEAR	310	40	8	5	CALCITE/ROCK BRECCIA
SHEAR	80	70	8	5	CALCITE/ROCK BRECCIA
BEDDING	300	65	9	10	NO FILL
JOINT	250	87	9	5	CALCITE VEIN
JOINT	75	70	9	5	THIN CALCITE COATING
JOINT	70	60	9	10	THIN CALCITE COATING
JOINT	200	70	9	15	THIN CALCITE COATING
JOINT	65	70	9	10	THIN CALCITE COATING
JOINT	350	45	9	15	
SHEAR	320	52	9	5	CALCITE/ROCK BRECCIA
SHEAR	50	87	9	5	THIN CALCITE COATING
JOINT	110	72	10	15	THIN CALCITE COATING
JOINT	300	60	10	20	NO FILL
JOINT	155	80	10	10	THIN CALCITE COATING
JOINT	190	77	10	20	NO FILL
JOINT	330	55	10	10	CALCITE VEIN
SHEAR	50	88	10	5	THIN CALCITE COATING
JOINT	215	68	11	20	NO FILL
JOINT	80	90	11	10	THIN CALCITE COATING
JOINT	65	59	11	5	THIN CALCITE COATING
JOINT	75	65	11	5	THIN CALCITE COATING
JOINT	60	72	11	15	THIN CALCITE COATING
JOINT	80	62	11	5	THIN CALCITE COATING
JOINT	315	34	11	20	NO FILL
BEDDING	310	45	12	10	THIN CALCITE COATING
JOINT	155	50	12	5	THIN CALCITE COATING
JOINT	160	75	12	5	THIN CALCITE COATING
JOINT	70	75	12	15	THIN CALCITE COATING
JOINT	70	80	12	15	THIN CALCITE COATING
JOINT	30	65	12	20	THIN CALCITE COATING
JOINT	270	90	12	20	THIN CALCITE COATING
JOINT	140	42	12	10	THIN CALCITE COATING
JOINT	135	77	12	10	THIN CALCITE COATING
SHEAR	20	85	12	5	CALCITE VEIN
BEDDING	330	32	13	10	THIN CALCITE COATING
JOINT	25	85	13	15	THIN CALCITE COATING
JOINT	100	84	13	15	THIN CALCITE COATING
SHEAR	40	85	13	10	CALCITE/ROCK BRECCIA
SHEAR	160	57	13	5	CALCITE VEIN
BEDDING	325	58	14	15	THIN CALCITE COATING
JOINT	40	90	14	20	THIN CALCITE COATING
JOINT	215	80	14	15	THIN CALCITE COATING

Type	Strike	Dip (RHR)	Station	Roughness (JRC)	Surface Coating
JOINT	325	90	14	10	THIN CALCITE COATING
JOINT	70	64	14	15	THIN CALCITE COATING
JOINT	220	65	14	10	THIN CALCITE COATING
JOINT	95	72	14	20	THIN CALCITE COATING
SHEAR	230	85	14	5	THIN CALCITE COATING
JOINT	120	75	15	15	THIN CALCITE COATING
JOINT	220	70	15	10	THIN CALCITE COATING
JOINT	140	55	15	10	THIN CALCITE COATING
JOINT	170	65	15	10	THIN CALCITE COATING
JOINT	10	85	15	15	THIN CALCITE COATING
JOINT	60	70	15	15	THIN CALCITE COATING
SHEAR	45	80	15	5	THIN CALCITE COATING
BEDDING	300	44	16	5	NO FILL
JOINT	215	78	16	15	THIN CALCITE COATING
JOINT	100	85	16	15	THIN CALCITE COATING
JOINT	40	85	16	15	THIN CALCITE COATING
JOINT	105	76	16	15	THIN CALCITE COATING
JOINT	50	75	16	10	THIN CALCITE COATING
JOINT	40	72	16	15	THIN CALCITE COATING
SHEAR	60	65	16	10	THIN CALCITE COATING
SHEAR	220	68	16	10	THIN CALCITE COATING
SHEAR	230	60	16	10	THIN CALCITE COATING
SHEAR	345	60	16	10	THIN CALCITE COATING
JOINT	50	60	17	10	THIN CALCITE COATING
SHEAR	45	85	17	5	CALCITE VEIN
JOINT	30	60	18	5	THIN CALCITE COATING
JOINT	170	80	18	10	THIN CALCITE COATING
JOINT	160	70	18	5	THIN CALCITE COATING
JOINT	0	32	18	15	THIN CALCITE COATING
JOINT	275	90	18	10	NO FILL
JOINT	70	78	18	10	THIN CALCITE COATING
JOINT	340	65	18	10	THIN CALCITE COATING
JOINT	140	74	18	15	THIN CALCITE COATING
JOINT	210	72	18	10	THIN CALCITE COATING
JOINT	300	45	18	10	THIN CALCITE COATING
JOINT	150	60	18	15	THIN CALCITE COATING
JOINT	270	80	18	10	THIN CALCITE COATING
JOINT	260	90	18	10	THIN CALCITE COATING
JOINT	245	75	18	10	THIN CALCITE COATING
JOINT	185	65	18	10	THIN CALCITE COATING
JOINT	310	45	18	10	THIN CALCITE COATING
JOINT	105	84	18	10	THIN CALCITE COATING
JOINT	240	85	18	10	NO FILL
SHEAR	205	72	18	15	THIN CALCITE COATING
BEDDING	290	50	19	5	NO FILL
JOINT	295	80	19	5	THIN CALCITE COATING
JOINT	185	55	19	10	THIN CALCITE COATING
JOINT	150	45	19	15	THIN CALCITE COATING

Type	Strike	Dip (RHR)	Station	Roughness (JRC)	Surface Coating
JOINT	310	90	19	15	NO FILL
JOINT	115	58	19	15	THIN CALCITE COATING
JOINT	90	70	19	15	THIN CALCITE COATING
JOINT	85	76	19	10	THIN CALCITE COATING
BEDDING	350	60	20	15	THIN CALCITE COATING
JOINT	80	45	20	5	THIN CALCITE COATING
JOINT	95	55	20	5	THIN CALCITE COATING
JOINT	170	40	20	15	THIN CALCITE COATING
JOINT	180	60	20	20	THIN CALCITE COATING
JOINT	110	40	20	5	THIN CALCITE COATING
JOINT	140	55	20	10	THIN CALCITE COATING
JOINT	210	58	20	20	THIN CALCITE COATING
JOINT	30	90	20	15	THIN CALCITE COATING
BEDDING	320	60	21	15	NO FILL
JOINT	320	56	21	10	THIN CALCITE COATING
JOINT	205	66	21	10	THIN CALCITE COATING
JOINT	205	80	21	15	THIN CALCITE COATING
JOINT	110	65	21	15	THIN CALCITE COATING
JOINT	105	56	21	15	THIN CALCITE COATING
JOINT	135	70	21	15	THIN CALCITE COATING
JOINT	205	75	21	15	THIN CALCITE COATING
JOINT	100	74	21	15	THIN CALCITE COATING
JOINT	80	20	21	20	NO FILL
SHEAR	20	65	21	5	THIN CALCITE COATING
BEDDING	300	55	22	5	THIN CALCITE COATING
JOINT	145	72	22	5	THIN CALCITE COATING
JOINT	290	90	22	20	THIN CALCITE COATING
JOINT	80	58	22	10	THIN CALCITE COATING
JOINT	210	50	22	15	THIN CALCITE COATING
JOINT	260	90	22	15	THIN CALCITE COATING
SHEAR	80	80	22	5	THIN CALCITE COATING
BEDDING	295	42	23	5	NO FILL
JOINT	160	82	23	15	THIN CALCITE COATING
JOINT	150	59	23	15	THIN CALCITE COATING
JOINT	260	90	23	10	THIN CALCITE COATING
JOINT	198	70	23	15	THIN CALCITE COATING
JOINT	255	85	23	5	CALCITE VEIN
JOINT	100	85	24	5	THIN CALCITE COATING
JOINT	15	84	24	15	THIN CALCITE COATING
JOINT	185	74	24	15	THIN CALCITE COATING
JOINT	155	70	25	15	THIN CALCITE COATING
JOINT	150	85	25	15	THIN CALCITE COATING
JOINT	140	38	25	15	THIN CALCITE COATING
BEDDING	325	60	26	10	THIN CALCITE COATING
JOINT	120	79	26	10	THIN CALCITE COATING
JOINT	175	59	26	15	THIN CALCITE COATING
JOINT	185	46	26	15	THIN CALCITE COATING
JOINT	20	80	26	15	THIN CALCITE COATING

Type	Strike	Dip (RHR)	Station	Roughness (JRC)	Surface Coating
JOINT	158	58	26	15	THIN CALCITE COATING
BEDDING	295	35	27	15	THIN CALCITE COATING
JOINT	285	90	27	20	THIN CALCITE COATING
JOINT	170	75	27	15	THIN CALCITE COATING
JOINT	135	70	27	10	THIN CALCITE COATING
JOINT	195	90	27	20	THIN CALCITE COATING
JOINT	235	75	27	15	THIN CALCITE COATING
BEDDING	320	58	28	15	THIN CALCITE COATING
JOINT	100	80	28	20	THIN CALCITE COATING
JOINT	30	45	28	20	THIN CALCITE COATING
JOINT	190	60	28	20	THIN CALCITE COATING
BEDDING	315	55	29	10	THIN CALCITE COATING
JOINT	190	82	29	5	CALCITE VEIN
JOINT	180	84	29	5	THIN CALCITE COATING
JOINT	195	64	29	5	CALCITE VEIN
JOINT	330	86	30	5	THIN CALCITE COATING
JOINT	220	68	30	15	THIN CALCITE COATING
JOINT	220	64	30	20	THIN CALCITE COATING
JOINT	95	59	30	20	THIN CALCITE COATING
JOINT	127	86	30	10	THIN CALCITE COATING
JOINT	190	64	30	15	THIN CALCITE COATING
JOINT	270	40	30	15	THIN CALCITE COATING
JOINT	275	40	30	15	THIN CALCITE COATING
JOINT	235	60	30	15	CALCITE VEIN
SHEAR	150	90	30	5	CALCITE/ROCK BRECCIA
JOINT	60	65	31	20	THIN CALCITE COATING
JOINT	200	82	31	10	THIN CALCITE COATING
JOINT	285	80	31	15	THIN CALCITE COATING
JOINT	275	80	31	20	THIN CALCITE COATING
JOINT	222	90	31	20	THIN CALCITE COATING
JOINT	150	50	31	5	THIN CALCITE COATING
SHEAR	355	65	31	5	CALCITE/ROCK BRECCIA
JOINT	235	63	32	15	THIN CALCITE COATING
JOINT	160	60	32	15	THIN CALCITE COATING
JOINT	150	76	32	15	THIN CALCITE COATING
JOINT	160	40	32	15	THIN CALCITE COATING
JOINT	305	72	32	5	CALCITE VEIN
SHEAR	10	84	32	5	CALCITE/ROCK BRECCIA
BEDDING	280	50	33	10	NO FILL
JOINT	190	60	33	15	THIN CALCITE COATING
JOINT	220	75	33	15	THIN CALCITE COATING
JOINT	350	40	33	15	THIN CALCITE COATING
JOINT	180	75	33	15	THIN CALCITE COATING
SHEAR	140	60	33	5	THIN CALCITE COATING
SHEAR	144	72	33	5	THIN CALCITE COATING
SHEAR	170	78	33	5	CALCITE VEIN
BEDDING	290	52	34	20	THIN CALCITE COATING
JOINT	190	90	34	20	THIN CALCITE COATING

Type	Strike	Dip (RHR)	Station	Roughness (JRC)	Surface Coating
JOINT	190	90	34	20	THIN CALCITE COATING
JOINT	110	70	34	10	THIN CALCITE COATING
JOINT	115	72	34	20	THIN CALCITE COATING
JOINT	125	35	34	20	THIN CALCITE COATING
JOINT	350	45	34	20	THIN CALCITE COATING
JOINT	95	40	34	15	THIN CALCITE COATING
JOINT	195	45	34	20	THIN CALCITE COATING
SHEAR	200	80	34	5	THIN CALCITE COATING
BEDDING	270	45	35	20	THIN CALCITE COATING
JOINT	60	65	35	5	THIN CALCITE COATING
JOINT	60	60	35	5	THIN CALCITE COATING
JOINT	40	85	35	10	THIN CALCITE COATING
BEDDING	280	35	36	10	THIN CALCITE COATING
JOINT	168	85	36	10	THIN CALCITE COATING
JOINT	168	79	36	10	THIN CALCITE COATING
JOINT	65	80	36	15	THIN CALCITE COATING
JOINT	170	90	36	15	THIN CALCITE COATING
JOINT	180	67	36	15	THIN CALCITE COATING
JOINT	28	90	36	15	THIN CALCITE COATING
JOINT	195	80	36	15	THIN CALCITE COATING
JOINT	144	41	36	15	THIN CALCITE COATING
JOINT	195	81	36	10	THIN CALCITE COATING
JOINT	290	50	37	15	NO FILL
JOINT	280	55	37	15	NO FILL
JOINT	170	50	37	15	THIN CALCITE COATING
JOINT	20	70	37	20	THIN CALCITE COATING
JOINT	145	87	37	5	THIN CALCITE COATING
JOINT	20	90	37	10	THIN CALCITE COATING
JOINT	275	75	38	5	THIN CALCITE COATING
JOINT	322	60	38	5	NO FILL
JOINT	22	60	38	10	THIN CALCITE COATING
JOINT	295	70	39	5	THIN CALCITE COATING
JOINT	145	80	39	5	THIN CALCITE COATING
JOINT	295	85	39	15	THIN CALCITE COATING
JOINT	178	70	39	20	THIN CALCITE COATING
JOINT	280	60	40	15	THIN CALCITE COATING
JOINT	190	70	40	10	THIN CALCITE COATING
BEDDING	280	45	40	15	NO FILL
JOINT	300	63	40	5	CALCITE/ROCK BRECCIA
JOINT	70	60	41	20	THIN CALCITE COATING
JOINT	175	85	41	10	NO FILL
JOINT	180	65	41	5	NO FILL
JOINT	310	62	41	5	THIN CALCITE COATING
JOINT	189	80	42	10	OBSERVED REMOTELY
JOINT	270	85	42	10	OBSERVED REMOTELY
JOINT	282	65	42	10	OBSERVED REMOTELY
JOINT	202	78	43	10	OBSERVED REMOTELY
JOINT	180	60	44	10	OBSERVED REMOTELY

Type	Strike	Dip (RHR)	Station	Roughness (JRC)	Surface Coating
JOINT	155	55	44	10	OBSERVED REMOTELY
JOINT	102	90	45	10	OBSERVED REMOTELY
JOINT	142	71	45	10	OBSERVED REMOTELY
JOINT	139	60	46	15	OBSERVED REMOTELY
JOINT	130	65	46	15	OBSERVED REMOTELY
JOINT	108	50	47	5	OBSERVED REMOTELY
BEDDING	280	40	48	15	THIN CALCITE COATING
JOINT	180	60	48	15	THIN CALCITE COATING
SHEAR	335	85	48	5	CALCITE VEIN
JOINT	320	90	48	10	CALCITE VEIN
JOINT	170	60	49	15	THIN CALCITE COATING
SHEAR	25	80	49	5	CALCITE VEIN
BEDDING	275	50	49	15	THIN CALCITE COATING

APPENDIX B

Slope Stability Output Data

Results of SWedge Analyses

6261.1.003.01
April 11, 2005

Wedge W-1
South Face

Wedge 1 South Face Proposed Max Pit Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.66588
Wedge height(on slope)=350 ft
Wedge width(on upper face)=144.222 ft
Wedge volume=1.45971e+006 ft3
Wedge weight=120426 tons
Wedge area (Joint5)=37314.6 ft2
Wedge area (Joint1)=29635.1 ft2
Wedge area (slope)=35061.3 ft2
Wedge area (upper face)=12511.8 ft2
Normal force (Joint5)=61142.6 tons
Normal force (Joint1)=58195.2 tons
Driving force=85208.4 tons
Resisting force=141947 tons

Water Pressures/Forces:
Average pressure on fissures=0.227602 tons/ft2
Water force on Joint5=8492.89 tons
Water force on Joint1=6745.02 tons

Failure Mode:
Sliding on intersection line (joints 5&1)

Joint Sets 5&1 line of Intersection:
plunge=45.0364 deg, trend=332.167 deg
length=494.661 ft

Joint Set 5 Data:

dip=62 deg, dip direction=30 deg
cohesion=0.32 tons/ft2, friction angle=45 deg

Joint Set 1 Data:

dip=65 deg, dip direction=270 deg
cohesion=0.36 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=YES
Overhanging slope face=NO
Externally applied force=NO

Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=340 deg

Water Pressure Data:

Water unit weight=0.031214 tons/ft3
Pressure definition method=Percent Filled Fissures
Percent Filled=50 %

Wedge 1 South Face Proposed Flooded Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.94755
Wedge height(on slope)=350 ft
Wedge width(on upper face)=135.797 ft
Wedge volume=1.29415e+006 ft3
Wedge weight=66390.1 tons
Wedge area (joint5)=34817.1 ft2
Wedge area (joint1)=27904 ft2
Wedge area (slope)=33013.2 ft2
Wedge area (upper face)=11092.8 ft2
Normal force (joint5)=37782.3 tons
Normal force (joint1)=36063.7 tons
Driving force=47508 tons
Resisting force=92524.1 tons

Failure Mode:
Sliding on intersection line (joints 5&1)

Joint Sets 5&1 line of Intersection:
plunge=45.6914 deg, trend=331.465 deg
length=489.108 ft

Joint Set 5 Data:

dip=63 deg, dip direction=30 deg
cohesion=0.28 tons/ft2, friction angle=45 deg

Joint Set 1 Data:

dip=65 deg, dip direction=270 deg
cohesion=0.32 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0513 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=340 deg

Wedge 1 South Face Proposed Flooded Pseudo-Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.24333
Wedge height(on slope)=350 ft
Wedge width(on upper face)=135.797 ft
Wedge volume=1.29415e+006 ft3
Wedge weight=106768 tons
Wedge area (joint5)=34817.1 ft2
Wedge area (joint1)=27904 ft2
Wedge area (slope)=33013.2 ft2
Wedge area (upper face)=11092.8 ft2
Normal force (joint5)=48934.4 tons
Normal force (joint1)=46708.5 tons
Driving force=90571.8 tons
Resisting force=112611 tons

Seismic Force:
Seismic force=20285.9 tons

Failure Mode:
Sliding on intersection line (joints 5&1)

Joint Sets 5&1 line of Intersection:
plunge=45.6914 deg, trend=331.465 deg
length=489.108 ft

Joint Set 5 Data:

dip=63 deg, dip direction=30 deg
cohesion=1 tons/ft2, friction angle=37 deg

Joint Set 1 Data:

dip=65 deg, dip direction=270 deg
cohesion=0.5 tons/ft2, friction angle=30 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=340 deg

Seismic Data:

Seismic coefficient=0.19
Direction=line of interesection J1&J2 but horizontal
trend=331.465 deg, plunge=0 deg

Wedge W-2
South Face

Wedge 2 South Face Proposed Max Pit, Static

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.94339
Wedge height(on slope)=350 ft
Wedge width(on upper face)=207.078 ft
Wedge volume=2.91168e+006 ft3
Wedge weight=240213 tons
Wedge area (Joint5)=43998.1 ft2
Wedge area (Joint1)=54624.4 ft2
Wedge area (slope)=48708 ft2
Wedge area (upper face)=24957.2 ft2
Normal force (Joint5)=130864 tons
Normal force (Joint1)=128445 tons
Driving force=154473 tons
Resisting force=300200 tons

Water Pressures/Forces:
Average pressure on fissures=0.227602 tons/ft2
Water force on Joint5=10014.1 tons
Water force on Joint1=12432.6 tons

Failure Mode:
Sliding on intersection line (joints 5&1)

Joint Sets 1&2 line of Intersection:
plunge=40.0207 deg, trend=351 deg
length=544.269 ft

Joint Set 5 Data:

dip=60 deg, dip direction=52 deg
cohesion=0.51 tons/ft2, friction angle=44 deg

Joint Set 1 Data:

dip=60 deg, dip direction=290 deg
cohesion=0.42 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=YES
Overhanging slope face=NO
Externally applied force=NO

Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=340 deg

Water Pressure Data:

Water unit weight=0.031214 tons/ft3
Pressure definition method=Percent Filled Fissures
Percent Filled=50 %

Wedge 2 South Face Proposed Flooded, Static:

Job Title:
SWEDGE - Surface Wedge Stability Analysis

Analysis Results:

Analysis type=Deterministic
Safety Factor=2.20624
Wedge height(on slope)=350 ft
Wedge width(on upper face)=207.078 ft
Wedge volume=2.91168e+006 ft3
Wedge weight=149369 tons
Wedge area (joint5)=43998.1 ft2
Wedge area (joint1)=54624.4 ft2
Wedge area (slope)=48708 ft2
Wedge area (upper face)=24957.2 ft2
Normal force (joint5)=87600.3 tons
Normal force (joint1)=87600.3 tons
Driving force=96053.9 tons
Resisting force=211918 tons

Failure Mode:
Sliding on intersection line (joints 5&1)

Joint Sets 5&1 line of Intersection:
plunge=40.0207 deg, trend=351 deg
length=544.269 ft

Joint Set 5 Data:

dip=60 deg, dip direction=52 deg
cohesion=0.4 tons/ft2, friction angle=45 deg

Joint Set 1 Data:

dip=60 deg, dip direction=290 deg
cohesion=0.35 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0513 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=340 deg

Wedge 2 South Face Proposed, Pseudo-Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.44395
Wedge height(on slope)=350 ft
Wedge width(on upper face)=207.078 ft
Wedge volume=2.91168e+006 ft3
Wedge weight=240213 tons
Wedge area (joint5=43998.1 ft2
Wedge area (joint1=54624.4 ft2
Wedge area (slope)=48708 ft2
Wedge area (upper face)=24957.2 ft2
Normal force (joint5=118401 tons
Normal force (joint1=118401 tons
Driving force=189425 tons
Resisting force=273520 tons

Seismic Force:
Seismic force=45640.5 tons

Failure Mode:
Sliding on intersection line (joints 5&1

Joint Sets 5&1 line of Intersection:
plunge=40.0207 deg, trend=351 deg
length=544.269 ft

Joint Set 5 ata:

dip=60 deg, dip direction=52 deg
cohesion=0.4 tons/ft2, friction angle=45 deg

Joint Set 1Data:

dip=60 deg, dip direction=290 deg
cohesion=0.35 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=340 deg

Seismic Data:

Seismic coefficient=0.19
Direction=line of interesection J5&J1 but horizontal
trend=351 deg, plunge=0 deg

Case 1
South Face

Case 1, South Wall, Proposed, Max Pit, Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.47157
Wedge height(on slope)=350 ft
Wedge width(on upper face)=157.235 ft
Wedge volume=2.67851e+006 ft3
Wedge weight=220977 tons
Wedge area (joint5)=51607.6 ft2
Wedge area (joint1)=41476.5 ft2
Wedge area (slope)=59011.4 ft2
Wedge area (upper face)=22958.7 ft2
Normal force (joint5)=70522.6 tons
Normal force (joint1)=113756 tons
Driving force=149138 tons
Resisting force=219467 tons

Water Pressures/Forces:
Average pressure on fissures=0.227602 tons/ft2
Water force on joint5=11746 tons
Water force on joint1=9440.13 tons

Failure Mode:
Sliding on intersection line (joints 5&1)

Joint Sets 5&2 line of Intersection:
plunge=42.4466 deg, trend=319.874 deg
length=518.593 ft

Joint Set 5 Data:

dip=60 deg, dip direction=18 deg
cohesion=0.28 tons/ft2, friction angle=45 deg

Joint Set 1 Data:

dip=50 deg, dip direction=280 deg
cohesion=0.5 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=YES
Overhanging slope face=NO

Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Water Pressure Data:

Water unit weight=0.031214 tons/ft3
Pressure definition method=Percent Filled Fissures
Percent Filled=50 %

Case 1, South Wall, Proposed, Flooded:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.66222
Wedge height(on slope)=350 ft
Wedge width(on upper face)=157.235 ft
Wedge volume=2.67851e+006 ft3
Wedge weight=137408 tons
Wedge area (joint5)=51607.6 ft2
Wedge area (joint1)=41476.5 ft2
Wedge area (slope)=59011.4 ft2
Wedge area (upper face)=22958.7 ft2
Normal force (joint5)=51156.1 tons
Normal force (joint1)=76605.8 tons
Driving force=92736.7 tons
Resisting force=154148 tons

Failure Mode:
Sliding on intersection line (joints 5&1)

Joint Sets 5&1 line of Intersection:
plunge=42.4466 deg, trend=319.874 deg
length=518.593 ft

Joint Set 5 Data:

dip=60 deg, dip direction=18 deg
cohesion=0.23 tons/ft2, friction angle=45 deg

Joint Set 1 Data:

dip=50 deg, dip direction=280 deg
cohesion=0.35 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0513 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Case 1, South Face, Flooded, Pseudo-Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.2131
Wedge height(on slope)=350 ft
Wedge width(on upper face)=157.235 ft
Wedge volume=2.67851e+006 ft3
Wedge weight=220977 tons
Wedge area (joint5)=51607.6 ft2
Wedge area (joint1)=41476.5 ft2
Wedge area (slope)=59011.4 ft2
Wedge area (upper face)=22958.7 ft2
Normal force (joint5)=67972.2 tons
Normal force (joint1)=101788 tons
Driving force=191124 tons
Resisting force=231852 tons

Seismic Force:
Seismic force=41985.6 tons

Failure Mode:
Sliding on intersection line (joints 5&1)

Joint Sets 5&1 line of Intersection:
plunge=42.4466 deg, trend=319.874 deg
length=518.593 ft

Joint Set 5 Data:

dip=60 deg, dip direction=18 deg
cohesion=0.23 tons/ft2, friction angle=45 deg

Joint Set 1 Data:

dip=50 deg, dip direction=280 deg
cohesion=0.35 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Seismic Data:

Seismic coefficient=0.19
Direction=line of interesection J5&J1 but horizontal
trend=319.874 deg, plunge=0 deg

Case 2
South Face

Case 2 South Face Proposed Max Pit Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.56689
Wedge height(on slope)=350 ft
Wedge width(on upper face)=109.033 ft
Wedge volume=862080 ft3
Wedge weight=71121.6 tons
Wedge area (joint1)=25786.8 ft2
Wedge area (Joint5)=25122.6 ft2
Wedge area (slope)=27389.3 ft2
Wedge area (upper face)=7389.25 ft2
Normal force (joint1)=23075.5 tons
Normal force (Joint5)=41530.7 tons
Driving force=50970.9 tons
Resisting force=79865.8 tons

Water Pressures/Forces:
Average pressure on fissures=0.227602 tons/ft2
Water force on joint1=5869.14 tons
Water force on Joint5=5717.97 tons

Failure Mode:
Sliding on intersection line (joints 1&5)

Joint Sets 1&5 line of Intersection:
plunge=45.7804 deg, trend=4.01713 deg
length=488.368 ft

Joint Set 1 Data:

dip=75 deg, dip direction=290 deg
cohesion=0.28 tons/ft2, friction angle=45 deg

Joint Set 5 Data:

dip=55 deg, dip direction=48 deg
cohesion=0.32 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3

Water pressures in the slope=YES
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Water Pressure Data:

Water unit weight=0.031214 tons/ft3
Pressure definition method=Percent Filled Fissures
Percent Filled=50 %

Case 2 South Face Proposed Flooded:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.86365
Wedge height(on slope)=350 ft
Wedge width(on upper face)=109.033 ft
Wedge volume=862080 ft3
Wedge weight=44224.7 tons
Wedge area (joint1)=25786.8 ft2
Wedge area (joint5)=25122.6 ft2
Wedge area (slope)=27389.3 ft2
Wedge area (upper face)=7389.25 ft2
Normal force (joint1)=17998.3 tons
Normal force (joint5)=29380.1 tons
Driving force=31694.6 tons
Resisting force=59067.6 tons

Failure Mode:
Sliding on intersection line (joints 1&5)

Joint Sets 1&5 line of Intersection:
plunge=45.7804 deg, trend=4.01713 deg
length=488.368 ft

Joint Set 1 Data:

dip=75 deg, dip direction=290 deg
cohesion=0.2 tons/ft2, friction angle=45 deg

Joint Set 5 Data:

dip=55 deg, dip direction=48 deg
cohesion=0.26 tons/ft2, friction angle=45 deg

Upper Face Data:

dip=0 deg, dip direction=195 deg

Case 2 South Face Proposed Flooded Pseudo-Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.20881
Wedge height(on slope)=350 ft
Wedge width(on upper face)=109.033 ft
Wedge volume=862080 ft3
Wedge weight=71121.6 tons
Wedge area (joint1)=25786.8 ft2
Wedge area (joint5)=25122.6 ft2
Wedge area (slope)=27389.3 ft2
Wedge area (upper face)=7389.25 ft2
Normal force (joint1)=23293.3 tons
Normal force (joint5)=38023.5 tons
Driving force=60395 tons
Resisting force=73006 tons

Seismic Force:
Seismic force=13513.1 tons

Failure Mode:
Sliding on intersection line (joints 1&5)

Joint Sets 1&5 line of Intersection:
plunge=45.7804 deg, trend=4.01713 deg
length=488.368 ft

Joint Set 1 Data:

dip=75 deg, dip direction=290 deg
cohesion=0.2 tons/ft2, friction angle=45 deg

Joint Set 5 Data:

dip=55 deg, dip direction=48 deg
cohesion=0.26 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Seismic Data:

Seismic coefficient=0.19
Direction=line of interesection J1&J5 but horizontal
trend=4.01713 deg, plunge=0 deg

Case 3
South Face

Case 3 South Wall Proposed Max Pit Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.91098
Wedge height(on slope)=350 ft
Wedge width(on upper face)=52.7066 ft
Wedge volume=187118 ft3
Wedge weight=15437.2 tons
Wedge area (joint1)=11334.1 ft2
Wedge area (joint5)=11334.1 ft2
Wedge area (slope)=12298.2 ft2
Wedge area (upper face)=1603.87 ft2
Normal force (joint1)=5236.93 tons
Normal force (joint5)=5236.93 tons
Driving force=12480.7 tons
Resisting force=23850.3 tons

Water Pressures/Forces:
Average pressure on fissures=0.227602 tons/ft2
Water force on joint1=2579.66 tons
Water force on joint5=2579.66 tons

Failure Mode:
Sliding on intersection line (joints 1&5)

Joint Set 1 Data:

dip=70 deg, dip direction=280 deg
cohesion=0.5 tons/ft2, friction angle=29 deg

Joint Set 5 Data:

dip=70 deg, dip direction=40 deg
cohesion=1 tons/ft2, friction angle=37 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=YES
Overhanging slope face=NO
Externally applied force=NO

Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Water Pressure Data:

Water unit weight=0.031214 tons/ft3
Pressure definition method=Percent Filled Fissures
Percent Filled=50 %

Case 3 South Wall Proposed Flooded Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=3.00977
Wedge height(on slope)=350 ft
Wedge width(on upper face)=52.7066 ft
Wedge volume=187118 ft3
Wedge weight=9599.16 tons
Wedge area (joint1)=11334.1 ft2
Wedge area (joint5)=11334.1 ft2
Wedge area (slope)=12298.2 ft2
Wedge area (upper face)=1603.87 ft2
Normal force (joint1)=4860.5 tons
Normal force (joint5)=4860.5 tons
Driving force=7760.72 tons
Resisting force=23358 tons

Failure Mode:
Sliding on intersection line (joints 1&5)

Joint Sets 1&5 line of Intersection:
plunge=53.9476 deg, trend=340 deg
length=432.912 ft

Joint Set 1 Data:

dip=70 deg, dip direction=280 deg
cohesion=0.5 tons/ft2, friction angle=29 deg

Joint Set 5 Data:

dip=70 deg, dip direction=40 deg
cohesion=1 tons/ft2, friction angle=37 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0513 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Case 3 South Wall Proposed Flooded Pseudo-Static:

Job Title:
SWEDGE - Surface Wedge Stability Analysis

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.72845
Wedge height(on slope)=350 ft
Wedge width(on upper face)=52.7066 ft
Wedge volume=187118 ft3
Wedge weight=15437.2 tons
Wedge area (joint1)=11334.1 ft2
Wedge area (joint5)=11334.1 ft2
Wedge area (slope)=12298.2 ft2
Wedge area (upper face)=1603.87 ft2
Normal force (joint1)=5776.38 tons
Normal force (joint5)=5776.38 tons
Driving force=14206.9 tons
Resisting force=24555.8 tons

Seismic Force:
Seismic force=2933.08 tons

Failure Mode:
Sliding on intersection line (joints 1&5)

Joint Sets 1&5 line of Intersection:
plunge=53.9476 deg, trend=340 deg
length=432.912 ft

Joint Set 1 Data:

dip=70 deg, dip direction=280 deg
cohesion=0.5 tons/ft2, friction angle=29 deg

Joint Set 5 Data:

dip=70 deg, dip direction=40 deg
cohesion=1 tons/ft2, friction angle=37 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Seismic Data:

Seismic coefficient=0.19
Direction=line of interesection J1&J5 but horizontal
trend=340 deg, plunge=0 deg

Case 4
South Face

Case 4 South Face, Proposed Max Pit Staic:

Analysis Results:

Analysis type=Deterministic
Safety Factor=2.24457
Wedge height(on slope)=350 ft
Wedge width(on upper face)=198.577 ft
Wedge volume=2.20276e+006 ft3
Wedge weight=181727 tons
Wedge area (Bedding)=36531.9 ft2
Wedge area (Bedding)=52582.9 ft2
Wedge area (slope)=38426.2 ft2
Wedge area (upper face)=18880.8 ft2
Normal force (Bedding)=95047.1 tons
Normal force (Bedding)=128466 tons
Driving force=119431 tons
Resisting force=268070 tons

Water Pressures/Forces:
Average pressure on fissures=0.227602 tons/ft2
Water force on Bedding=8314.74 tons
Water force on Bedding=11968 tons

Failure Mode:
Sliding on intersection line (joints 2&Bedding)

Joint Sets 2&Bedding line of Intersection:
plunge=41.0865 deg, trend=336.489 deg
length=532.564 ft

Joint Set 2 Data:

dip=75 deg, dip direction=260 deg
cohesion=0.5 tons/ft2, friction angle=45 deg

Joint Set Bedding Data:

dip=57 deg, dip direction=32 deg
cohesion=0.5 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=YES

Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Water Pressure Data:

Water unit weight=0.031214 tons/ft3
Pressure definition method=Percent Filled Fissures
Percent Filled=50 %

Case 4 South Wall Proposed Flooded Static:

Job Title:
SWEDGE - Surface Wedge Stability Analysis

Analysis Results:

Analysis type=Deterministic
Safety Factor=2.23823
Wedge height(on slope)=350 ft
Wedge width(on upper face)=198.577 ft
Wedge volume=2.20276e+006 ft3
Wedge weight=113001 tons
Wedge area (joint2)=36531.9 ft2
Wedge area (bedding)=52582.9 ft2
Wedge area (slope)=38426.2 ft2
Wedge area (upper face)=18880.8 ft2
Normal force (joint1)=64272.3 tons
Normal force (joint2)=87324.3 tons
Driving force=74264.2 tons
Resisting force=166220 tons

Failure Mode:
Sliding on intersection line (joints 2&bedding)

Joint Sets 2&bedding line of Intersection:
plunge=41.0865 deg, trend=336.489 deg
length=532.564 ft

Joint Set 2 Data:

dip=75 deg, dip direction=260 deg
cohesion=0.17 tons/ft2, friction angle=45 deg

Joint Set Bedding Data:

dip=57 deg, dip direction=32 deg
cohesion=0.16 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0513 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Case 4 Proposed Flooded, Pseudo-Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.49895
Wedge height(on slope)=350 ft
Wedge width(on upper face)=198.577 ft
Wedge volume=2.20276e+006 ft3
Wedge weight=181727 tons
Wedge area (joint2)=36531.9 ft2
Wedge area (bedding)=52582.9 ft2
Wedge area (slope)=38426.2 ft2
Wedge area (upper face)=18880.8 ft2
Normal force (joint2)=86238 tons
Normal force (bedding)=117168 tons
Driving force=145455 tons
Resisting force=218030 tons

Seismic Force:
Seismic force=34528.2 tons

Failure Mode:
Sliding on intersection line (joints 2&Bedding)

Joint Sets 2&Bedding line of Intersection:
plunge=41.0865 deg, trend=336.489 deg
length=532.564 ft

Joint Set 2 Data:

dip=75 deg, dip direction=260 deg
cohesion=0.17 tons/ft2, friction angle=45 deg

Joint Set Bedding Data:

dip=57 deg, dip direction=32 deg
cohesion=0.16 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3

Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Seismic Data:

Seismic coefficient=0.19
Direction=line of interesection J1&J2 but horizontal
trend=336.489 deg, plunge=0 deg

Case 5
South Face

Case 5 South Face Proposed Max Pit Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.44848
Wedge height(on slope)=350 ft
Wedge width(on upper face)=180.498 ft
Wedge volume=5.55659e+006 ft3
Wedge weight=458418 tons
Wedge area (Bedding)=130609 ft2
Wedge area (joint2)=33205.8 ft2
Wedge area (slope)=106642 ft2
Wedge area (upper face)=47627.9 ft2
Normal force (Bedding)=288100 tons
Normal force (joint2)=90618.8 tons
Driving force=308990 tons
Resisting force=447566 tons

Water Pressures/Forces:
Average pressure on fissures=0.227602 tons/ft2
Water force on Bedding=29726.9 tons
Water force on joint2=7557.72 tons

Failure Mode:
Sliding on intersection line (joints Bedding&2)

Joint Sets Bedding *&2 line of Intersection:*
plunge=42.3793 deg, trend=335.848 deg
length=519.26 ft

Joint Set Bedding Data:

dip=45 deg, dip direction=360 deg
cohesion=0.4 tons/ft2, friction angle=45 deg

Joint Set 2 Data:

dip=75 deg, dip direction=260 deg
cohesion=0.5 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet

rock unit weight=0.0825 tons/ft3
Water pressures in the slope=YES
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=290 deg

Water Pressure Data:

Water unit weight=0.031214 tons/ft3
Pressure definition method=Percent Filled Fissures
Percent Filled=50 %

Case 5 South Wall, Flooded, Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.63979
Wedge height(on slope)=350 ft
Wedge width(on upper face)=180.498 ft
Wedge volume=5.55659e+006 ft3
Wedge weight=285053 tons
Wedge area (bedding)=130609 ft2
Wedge area (joint2)=33205.8 ft2
Wedge area (slope)=106642 ft2
Wedge area (upper face)=47627.9 ft2
Normal force (bedding)=197631 tons
Normal force (joint2)=61048 tons
Driving force=192136 tons
Resisting force=315062 tons

Failure Mode:
Sliding on intersection line (Bedding&2)

Joint Sets bedding&2 line of Intersection:
plunge=42.3793 deg, trend=335.848 deg
length=519.26 ft

Joint Set Bedding Data:

dip=45 deg, dip direction=360 deg
cohesion=0.33 tons/ft2, friction angle=45 deg

Joint Set 2 Data:

dip=75 deg, dip direction=260 deg
cohesion=0.4 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0513 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=290 deg

Case 5 South Face Flooded Pseudo-Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.19263
Wedge height(on slope)=350 ft
Wedge width(on upper face)=180.498 ft
Wedge volume=5.55659e+006 ft3
Wedge weight=458418 tons
Wedge area (Bedding)=130609 ft2
Wedge area (joint2)=33205.8 ft2
Wedge area (slope)=106642 ft2
Wedge area (upper face)=47627.9 ft2
Normal force (Bedding)=317827 tons
Normal force (joint2)=98176.5 tons
Driving force=396090 tons
Resisting force=472387 tons

Seismic Force:
Seismic force=87099.5 tons

Failure Mode:
Sliding on intersection line (joints 1&2)

Joint Sets Bedding&2 line of Intersection:
plunge=42.3793 deg, trend=335.848 deg
length=519.26 ft

Joint Set Bedding Data:

dip=45 deg, dip direction=360 deg
cohesion=0.33 tons/ft2, friction angle=45 deg

Joint Set 2 Data:

dip=75 deg, dip direction=260 deg
cohesion=0.4 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=340 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=290 deg

Seismic Data:

Seismic coefficient=0.19
Direction=line of interesection J1&J2 But horizontal
trend=335.848 deg, plunge=42.3793 deg

Case 1
North Face

Case 1 North Wall Proposed Max Pit Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=2.08905
Wedge height(on slope)=350 ft
Wedge width(on upper face)=320.07 ft
Wedge volume=3.54296e+006 ft3
Wedge weight=292294 tons
Wedge area (Joint5)=68005.9 ft2
Wedge area (joint2)=60033.8 ft2
Wedge area (slope)=38345.2 ft2
Wedge area (upper face)=30368.2 ft2
Normal force (Joint5)=157796 tons
Normal force (joint2)=127471 tons
Driving force=162372 tons
Resisting force=339204 tons

Water Pressures/Forces:
Average pressure on fissures=1.82082 tons/ft2
Water force on Joint5=123826 tons
Water force on joint2=109311 tons

Failure Mode:
Sliding on intersection line (joints 5&2)

Joint Sets 5&2 line of Intersection:
plunge=33.7458 deg, trend=145.312 deg
length=630.053 ft

Joint Set 5 Data:

dip=60 deg, dip direction=78 deg
cohesion=0.44 tons/ft2, friction angle=45 deg

Joint Set 2 Data:

dip=75 deg, dip direction=225 deg
cohesion=0.4 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=150 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=YES
Overhanging slope face=NO

Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Water Pressure Data:

Water unit weight=0.031214 tons/ft3
Pressure definition method=Filled Fissures

Case 1 North Wall Proposed Flooded Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=3.7812
Wedge height(on slope)=350 ft
Wedge width(on upper face)=320.07 ft
Wedge volume=3.54296e+006 ft3
Wedge weight=181754 tons
Wedge area (joint5)=68005.9 ft2
Wedge area (joint2)=60033.8 ft2
Wedge area (slope)=38345.2 ft2
Wedge area (upper face)=30368.2 ft2
Normal force (joint5)=175118 tons
Normal force (joint2)=147235 tons
Driving force=100966 tons
Resisting force=381772 tons

Failure Mode:
Sliding on intersection line (joints 5&2)

Joint Sets 5&2 line of Intersection:
plunge=33.7458 deg, trend=145.312 deg
length=630.053 ft

Joint Set 5 Data:

dip=60 deg, dip direction=78 deg
cohesion=0.45 tons/ft2, friction angle=45 deg

Joint Set 2 Data:

dip=75 deg, dip direction=225 deg
cohesion=0.48 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=150 deg
slope height=350 feet
rock unit weight=0.0513 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Case 1 North Wall Proposed Flooded Pseudo-Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=2.45514
Wedge height(on slope)=350 ft
Wedge width(on upper face)=320.07 ft
Wedge volume=3.54296e+006 ft3
Wedge weight=292294 tons
Wedge area (joint5)=68005.9 ft2
Wedge area (joint2)=60033.8 ft2
Wedge area (slope)=38345.2 ft2
Wedge area (upper face)=30368.2 ft2
Normal force (joint5)=245875 tons
Normal force (joint2)=206726 tons
Driving force=208551 tons
Resisting force=512020 tons

Seismic Force:
Seismic force=55535.8 tons

Failure Mode:
Sliding on intersection line (joints 5&2)

Joint Sets 5&2 line of Intersection:
plunge=33.7458 deg, trend=145.312 deg
length=630.053 ft

Joint Set 5 Data:

dip=60 deg, dip direction=78 deg
cohesion=0.45 tons/ft2, friction angle=45 deg

Joint Set 2 Data:

dip=75 deg, dip direction=225 deg
cohesion=0.48 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=150 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Seismic Data:

Seismic coefficient=0.19
Direction=line of interesection J5&J2 but horizontal
trend=145.312 deg, plunge=0 deg

Case 2
North Face

Case 2 North Wall Proposed Max Pit Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.61485
Wedge height(on slope)=350 ft
Wedge width(on upper face)=23.8381 ft
Wedge volume=53048.3 ft3
Wedge weight=4376.49 tons
Wedge area (joint2)=5092.67 ft2
Wedge area (Joint1)=7993.72 ft2
Wedge area (slope)=7708.86 ft2
Wedge area (upper face)=454.7 ft2
Normal force (joint2)=3091.84 tons
Normal force (Joint1)=1773.75 tons
Driving force=3169.28 tons
Resisting force=5117.91 tons

Failure Mode:
Sliding on intersection line (joints 2&1)

Joint Sets 2&1 line of Intersection:
plunge=46.3991 deg, trend=197.33 deg
length=483.318 ft

Joint Set 2 Data:

dip=55 deg, dip direction=240 deg
cohesion=0.026 tons/ft2, friction angle=45 deg

Joint Set 1 Data:

dip=80 deg, dip direction=118 deg
cohesion=0.015 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=150 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Case 2 North Wall Proposed Flooded Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.63007
Wedge height(on slope)=350 ft
Wedge width(on upper face)=23.8381 ft
Wedge volume=53048.3 ft3
Wedge weight=2721.38 tons
Wedge area (joint2)=5092.67 ft2
Wedge area (joint1)=7993.72 ft2
Wedge area (slope)=7708.86 ft2
Wedge area (upper face)=454.7 ft2
Normal force (joint2)=1922.56 tons
Normal force (joint1)=1102.95 tons
Driving force=1970.72 tons
Resisting force=3212.4 tons

Failure Mode:
Sliding on intersection line (joints 2&1)

Joint Sets 2&1 line of Intersection:
plunge=46.3991 deg, trend=197.33 deg
length=483.318 ft

Joint Set 2 Data:

dip=55 deg, dip direction=240 deg
cohesion=0.021 tons/ft2, friction angle=45 deg

Joint Set 1 Data:

dip=80 deg, dip direction=118 deg
cohesion=0.01 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=150 deg
slope height=350 feet
rock unit weight=0.0513 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Case 2 North Face Flooded Pseudo-Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.26286
Wedge height(on slope)=350 ft
Wedge width(on upper face)=23.8381 ft
Wedge volume=53048.3 ft3
Wedge weight=4376.49 tons
Wedge area (joint2)=5092.67 ft2
Wedge area (joint1)=7993.72 ft2
Wedge area (slope)=7708.86 ft2
Wedge area (upper face)=454.7 ft2
Normal force (joint2)=3091.84 tons
Normal force (joint1)=1773.75 tons
Driving force=4000.81 tons
Resisting force=5052.48 tons

Seismic Force:
Seismic force=831.533 tons

Failure Mode:
Sliding on intersection line (joints 2&1)

Joint Sets 2&1 line of Intersection:
plunge=46.3991 deg, trend=197.33 deg
length=483.318 ft

Joint Set 2 Data:

dip=55 deg, dip direction=240 deg
cohesion=0.021 tons/ft2, friction angle=45 deg

Joint Set 1 Data:

dip=80 deg, dip direction=118 deg
cohesion=0.01 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=150 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Seismic Data:

Seismic coefficient=0.19
Direction=line of intersection J2&J1 but horizontal
trend=197.33 deg, plunge=46.3991 deg

Case 3
North Face

Case 3 North Face Proposed Max Pit Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.17233
Wedge height(on slope)=350 ft
Wedge width(on upper face)=83.6972 ft
Wedge volume=1.65783e+006 ft3
Wedge weight=136771 tons
Wedge area (joint5)=17783.3 ft2
Wedge area (joint3)=69085.7 ft2
Wedge area (slope)=68615.2 ft2
Wedge area (upper face)=14210 ft2
Normal force (joint5)=43964.8 tons
Normal force (joint3)=47172.7 tons
Driving force=102415 tons
Resisting force=120064 tons

Water Pressures/Forces:
Average pressure on fissures=0.227602 tons/ft2
Water force on joint1=4047.51 tons
Water force on joint2=15724.1 tons

Failure Mode:
Sliding on intersection line (joints 5&3)

Joint Sets 5&3 line of Intersection:
plunge=48.487 deg, trend=127.286 deg
length=467.411 ft

Joint Set 5 Data:

dip=60 deg, dip direction=78 deg
cohesion=0.5 tons/ft2, friction angle=45 deg

Joint Set 3 Data:

dip=55 deg, dip direction=165 deg
cohesion=0.29 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=150 deg
slope height=350 feet
rock unit weight=0.0825 tons/ft3
Water pressures in the slope=YES
Overhanging slope face=NO
Externally applied force=NO

Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Water Pressure Data:

Water unit weight=0.031214 tons/ft3
Pressure definition method=Percent Filled Fissures
Percent Filled=50 %

Case 3 North Face Proposed Flooded:

Job Title:
SWEDGE - Surface Wedge Stability Analysis

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.42739
Wedge height(on slope)=350 ft
Wedge width(on upper face)=83.6972 ft
Wedge volume=1.65783e+006 ft3
Wedge weight=85046.8 tons
Wedge area (joint5)=17783.3 ft2
Wedge area (joint3)=69085.7 ft2
Wedge area (slope)=68615.2 ft2
Wedge area (upper face)=14210 ft2
Normal force (joint5)=29854.9 tons
Normal force (joint3)=39110.4 tons
Driving force=63683.5 tons
Resisting force=90901.3 tons

Failure Mode:
Sliding on intersection line (joints 5&3)

Joint Sets 5&3 line of Intersection:
plunge=48.487 deg, trend=127.286 deg
length=467.411 ft

Joint Set 5 Data:

dip=60 deg, dip direction=78 deg
cohesion=0.34 tons/ft2, friction angle=45 deg

Joint Set 3 Data:

dip=55 deg, dip direction=165 deg
cohesion=0.23 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=150 deg
slope height=350 feet
rock unit weight=0.0513 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Case 3 North Face Proposed Flooded Pseudo-Static:

Analysis Results:

Analysis type=Deterministic
Safety Factor=1.15077
Wedge height(on slope)=350 ft
Wedge width(on upper face)=83.6972 ft
Wedge volume=1.65783e+006 ft3
Wedge weight=85046.8 tons
Wedge area (joint5)=17783.3 ft2
Wedge area (joint3)=69085.7 ft2
Wedge area (slope)=68615.2 ft2
Wedge area (upper face)=14210 ft2
Normal force (joint5)=29854.9 tons
Normal force (joint3)=39110.4 tons
Driving force=78991.9 tons
Resisting force=90901.3 tons

Seismic Force:
Seismic force=15308.4 tons

Failure Mode:
Sliding on intersection line (joints 5&3)

Joint Sets 5&3 line of Intersection:
plunge=48.487 deg, trend=127.286 deg
length=467.411 ft

Joint Set 5 Data:

dip=60 deg, dip direction=78 deg
cohesion=0.34 tons/ft2, friction angle=45 deg

Joint Set 3 Data:

dip=55 deg, dip direction=165 deg
cohesion=0.23 tons/ft2, friction angle=45 deg

Slope Data:

dip=60 deg, dip direction=150 deg
slope height=350 feet
rock unit weight=0.0513 tons/ft3
Water pressures in the slope=NO
Overhanging slope face=NO
Externally applied force=NO
Tension crack=NO

Upper Face Data:

dip=0 deg, dip direction=195 deg

Seismic Data:

Seismic coefficient=0.19
Direction=line of interesection J5&J3 but horizontal
trend=127.286 deg, plunge=0 deg

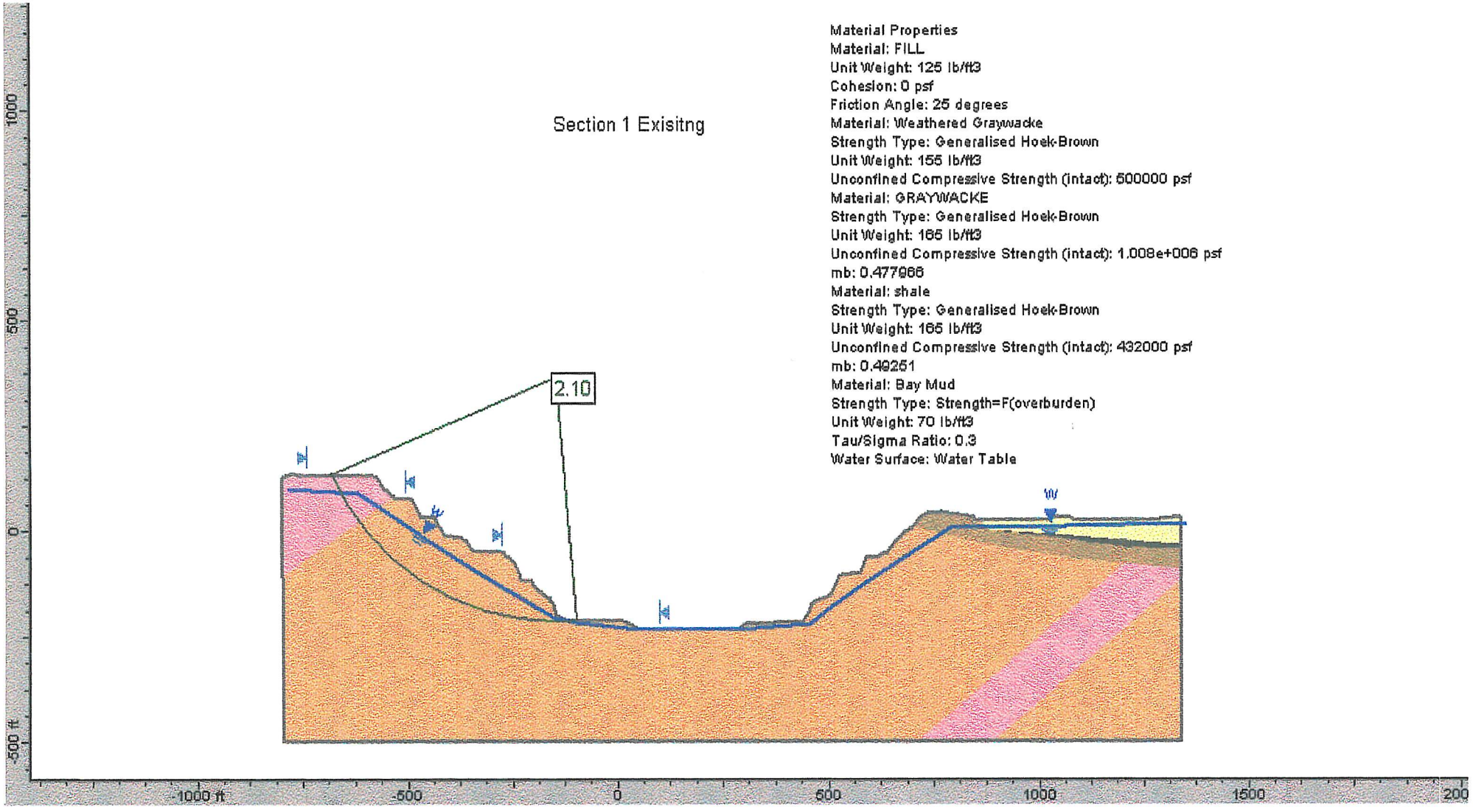
Circular Analyses

6261.1.003.01
April 11, 2005

Cross Section 1

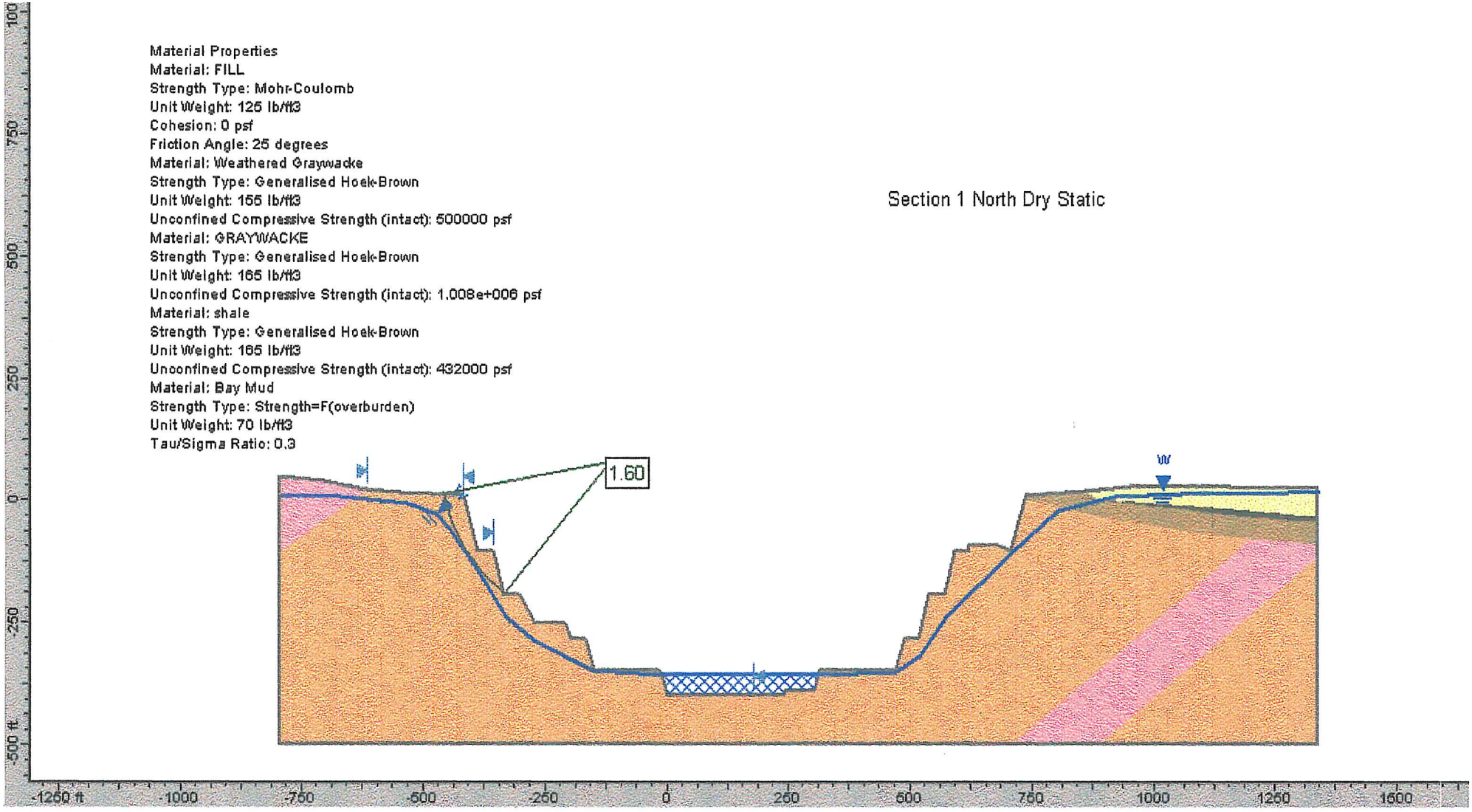
Section 1 Exisitng

Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
mb: 0.477966
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf
mb: 0.49251
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3
Water Surface: Water Table



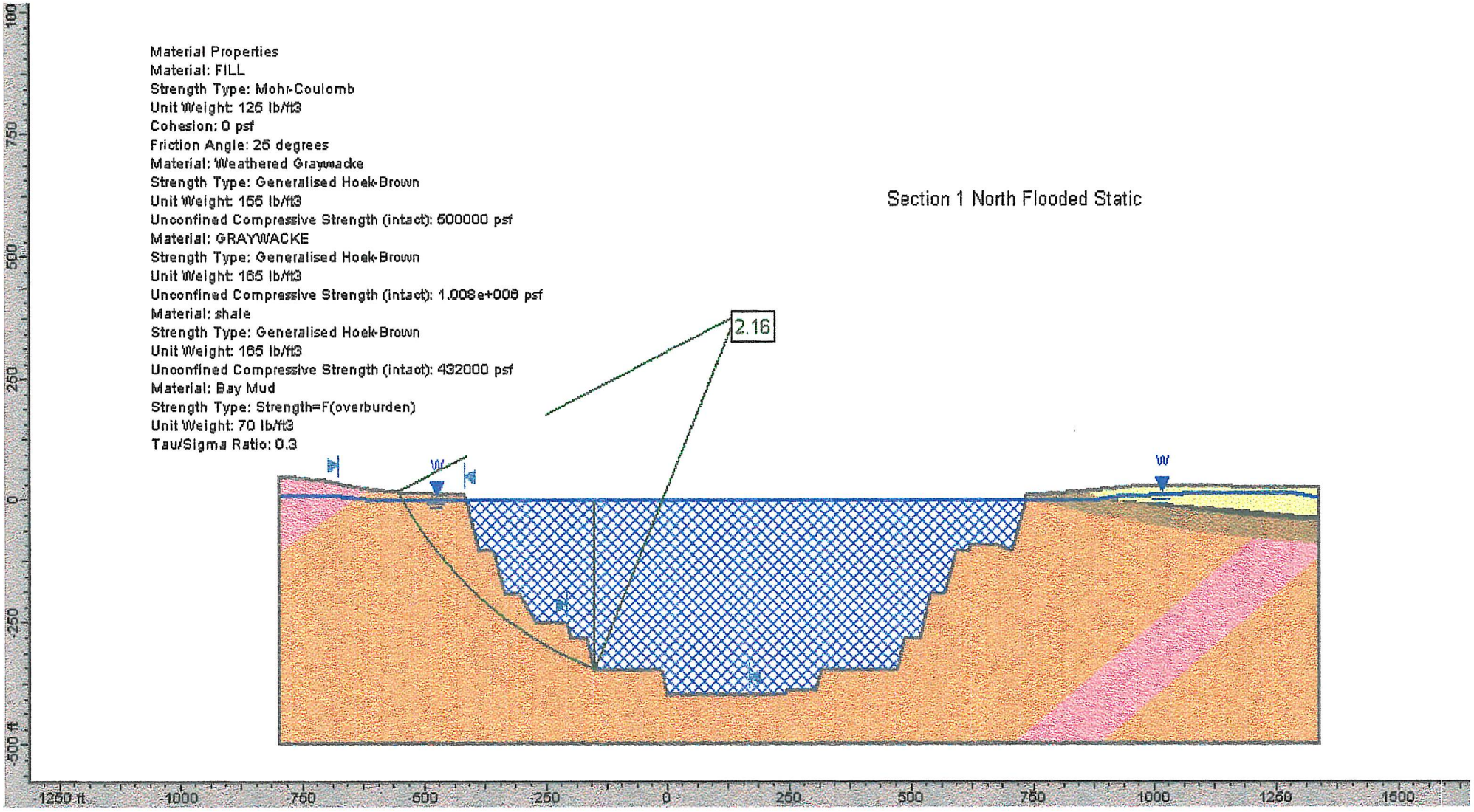
Material Properties
Material: FILL
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3

Section 1 North Dry Static



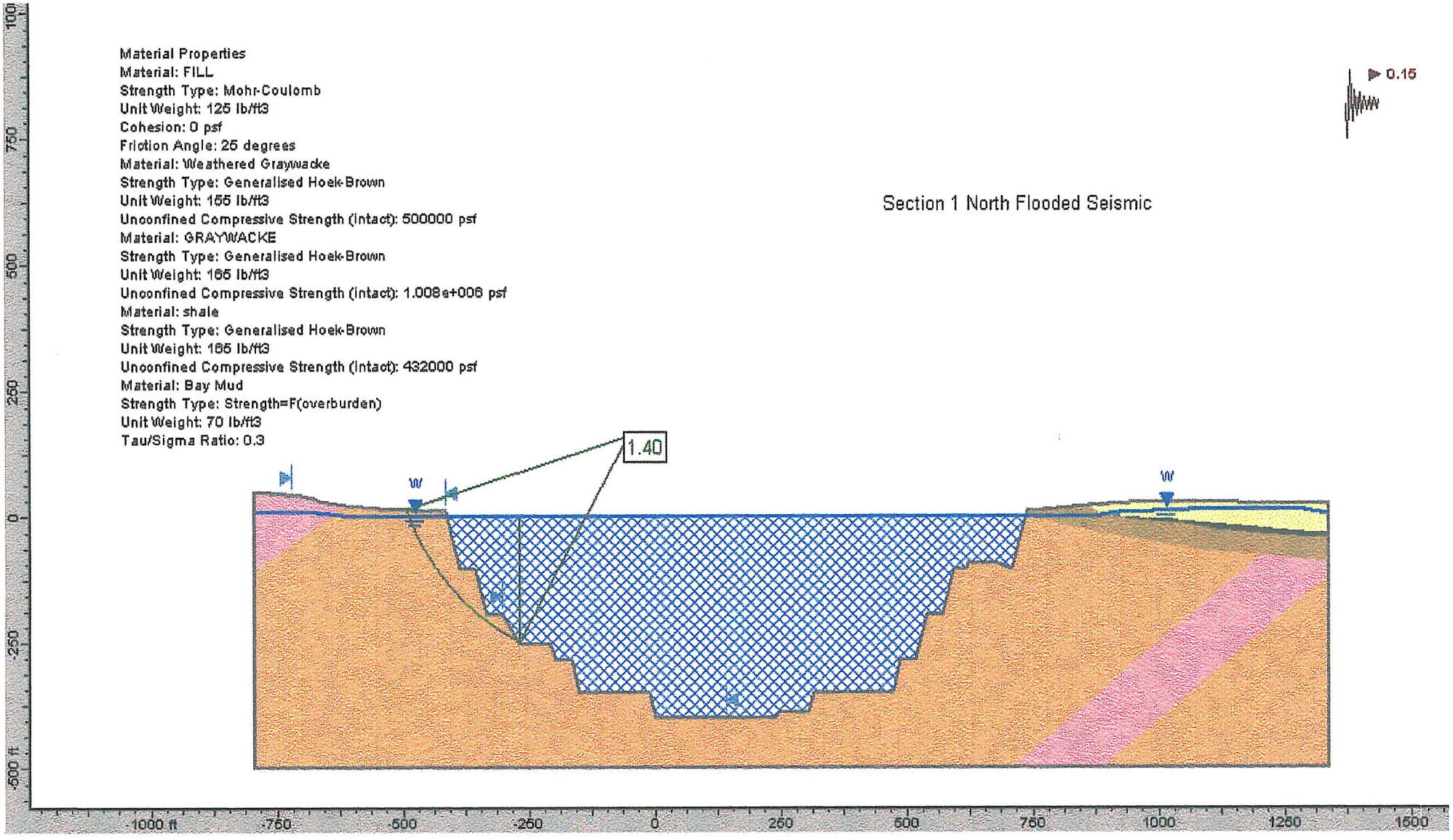
Material Properties
Material: FILL
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3

Section 1 North Flooded Static



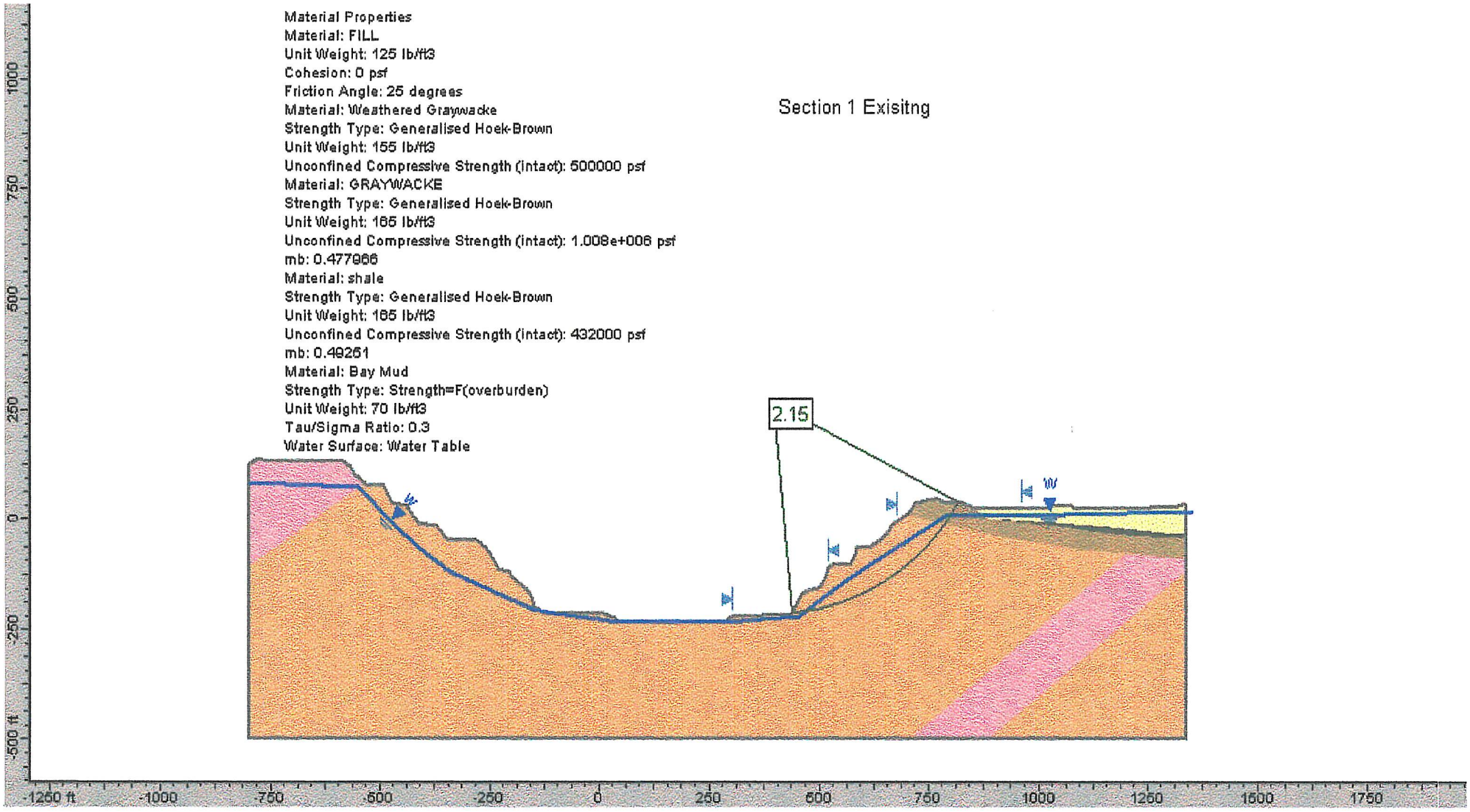
Material Properties
Material: FILL
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3

Section 1 North Flooded Seismic



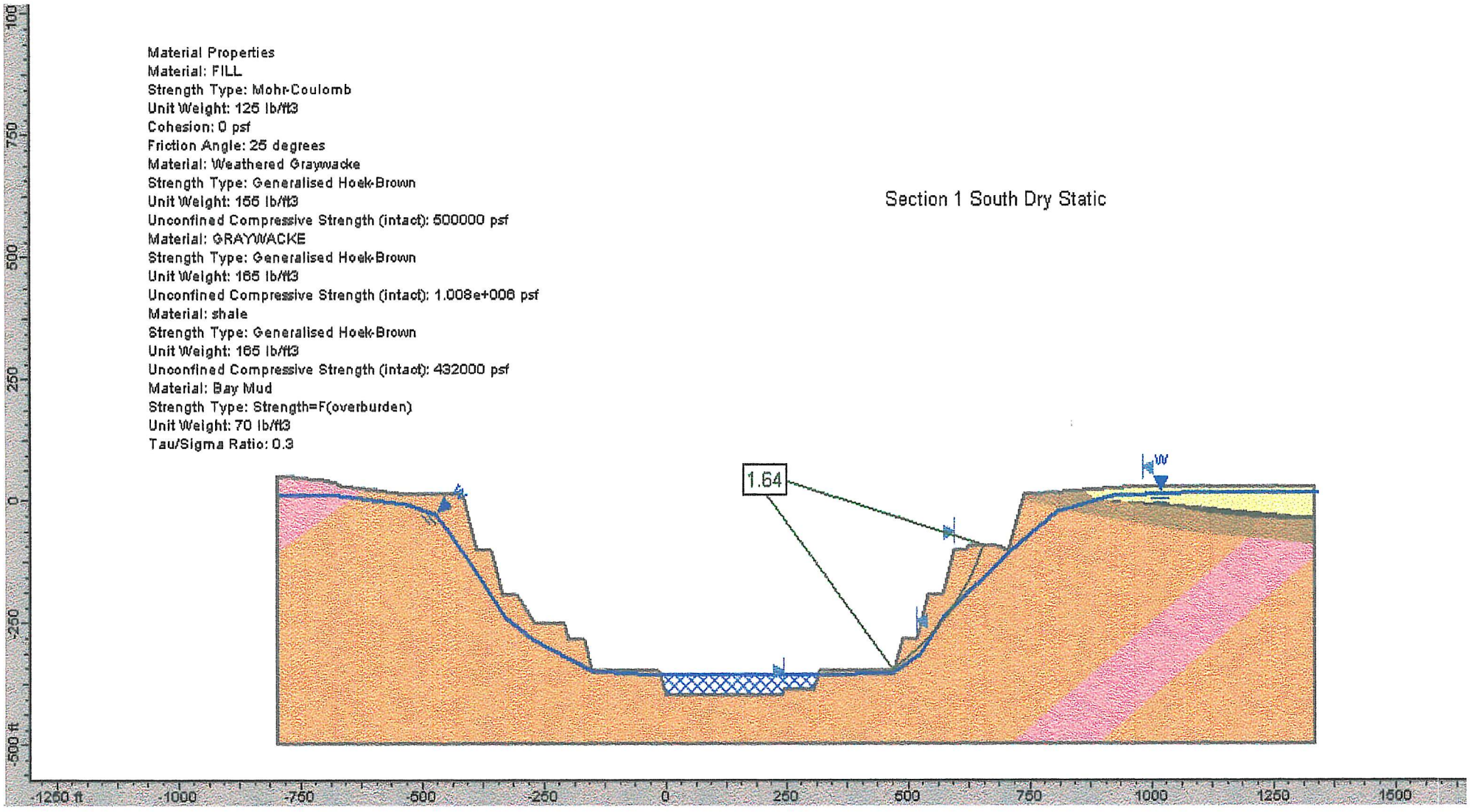
Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
mb: 0.477966
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf
mb: 0.49261
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3
Water Surface: Water Table

Section 1 Exisitng



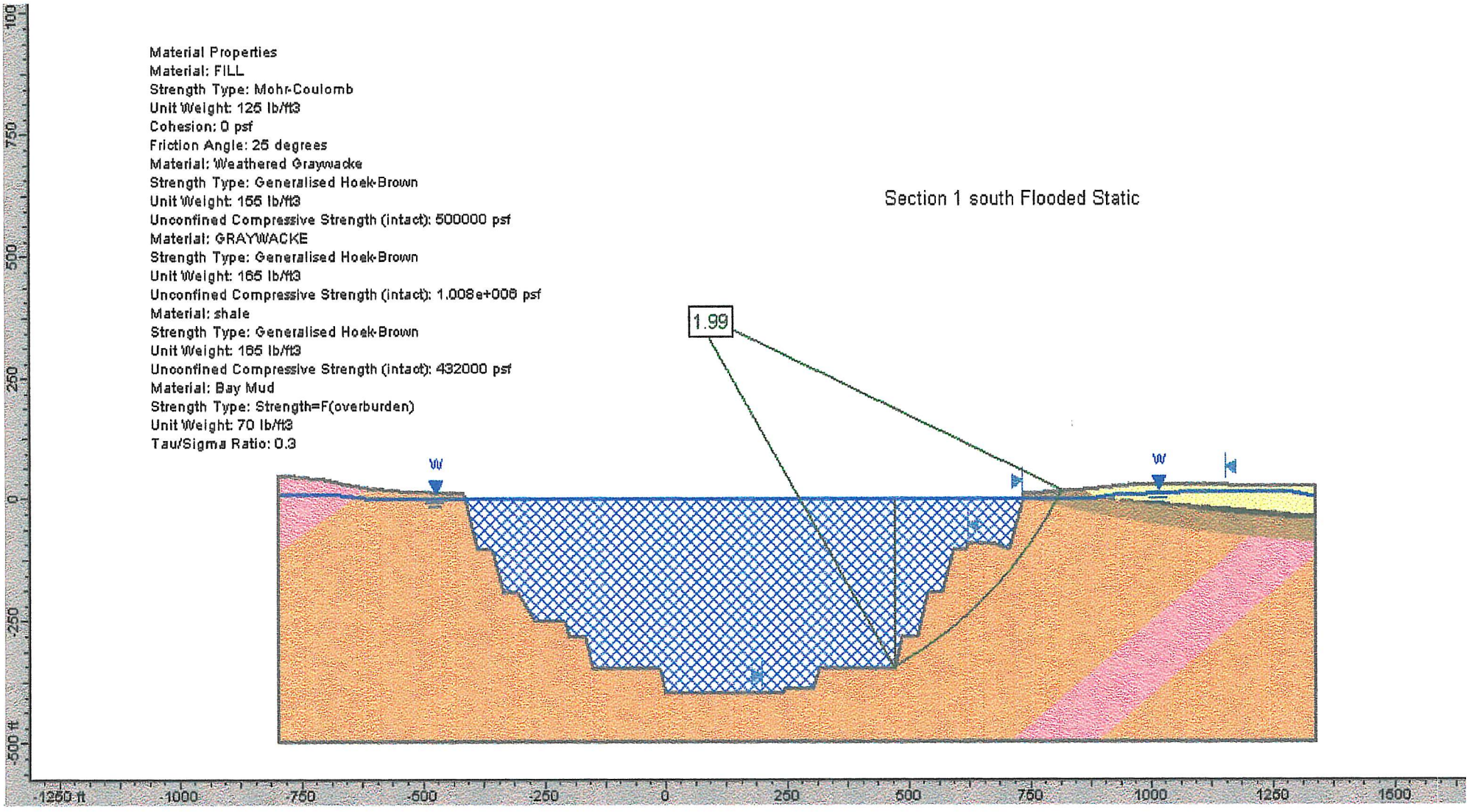
Material Properties
Material: FILL
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3

Section 1 South Dry Static

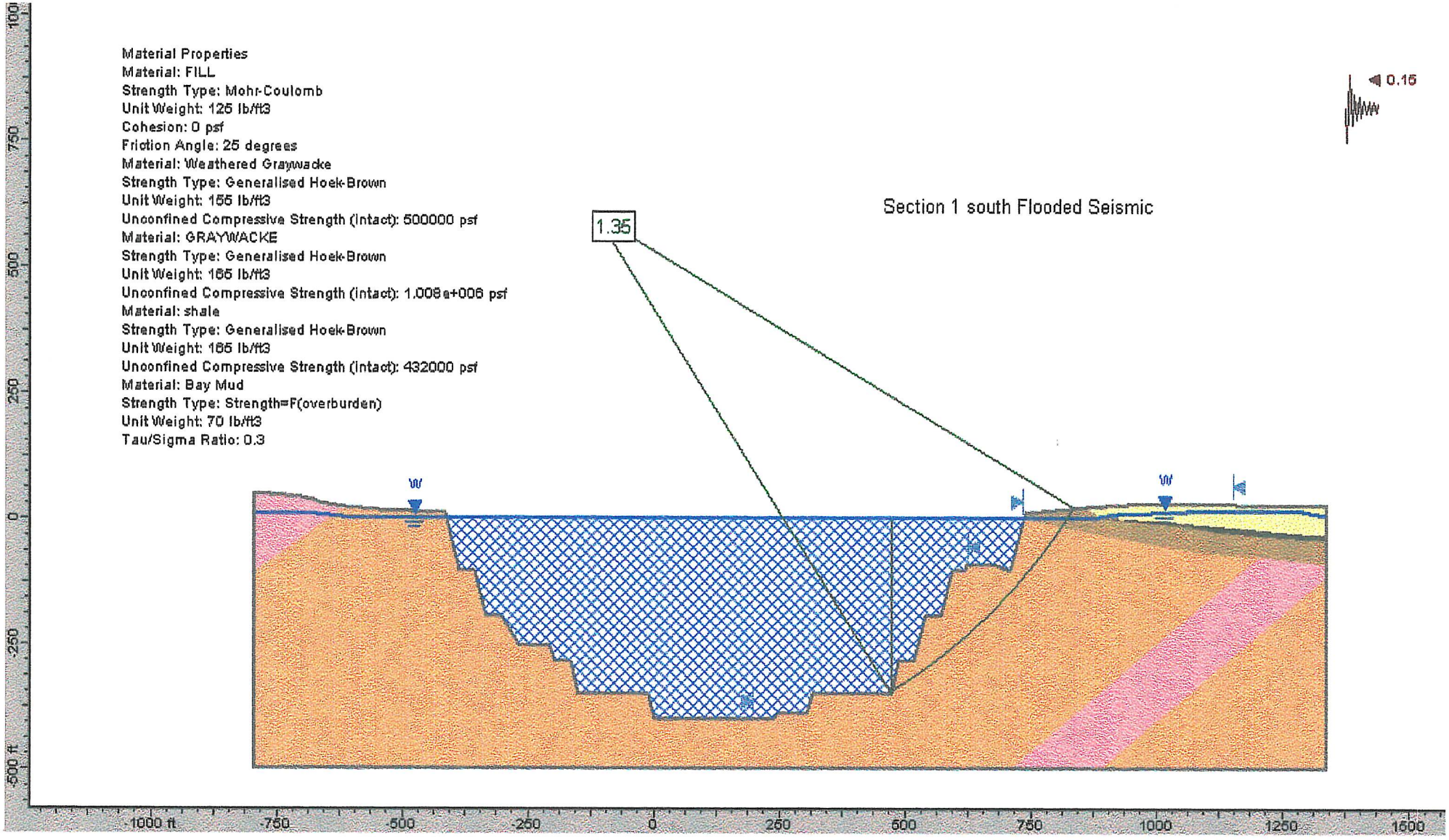


Material Properties
Material: FILL
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+008 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3

Section 1 south Flooded Static



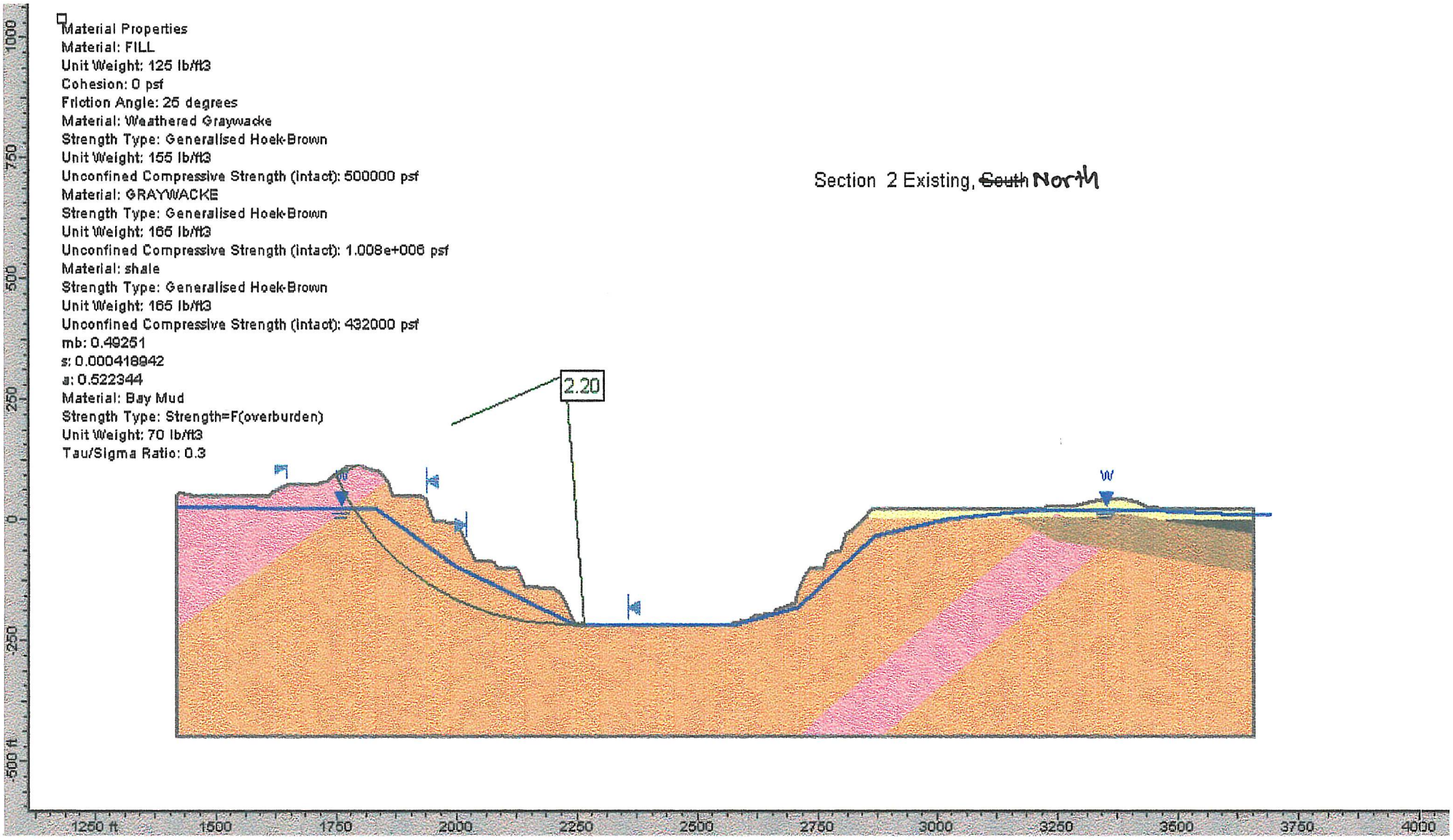
Material Properties
Material: FILL
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (Intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (Intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (Intact): 432000 psf
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3



Cross Section 2

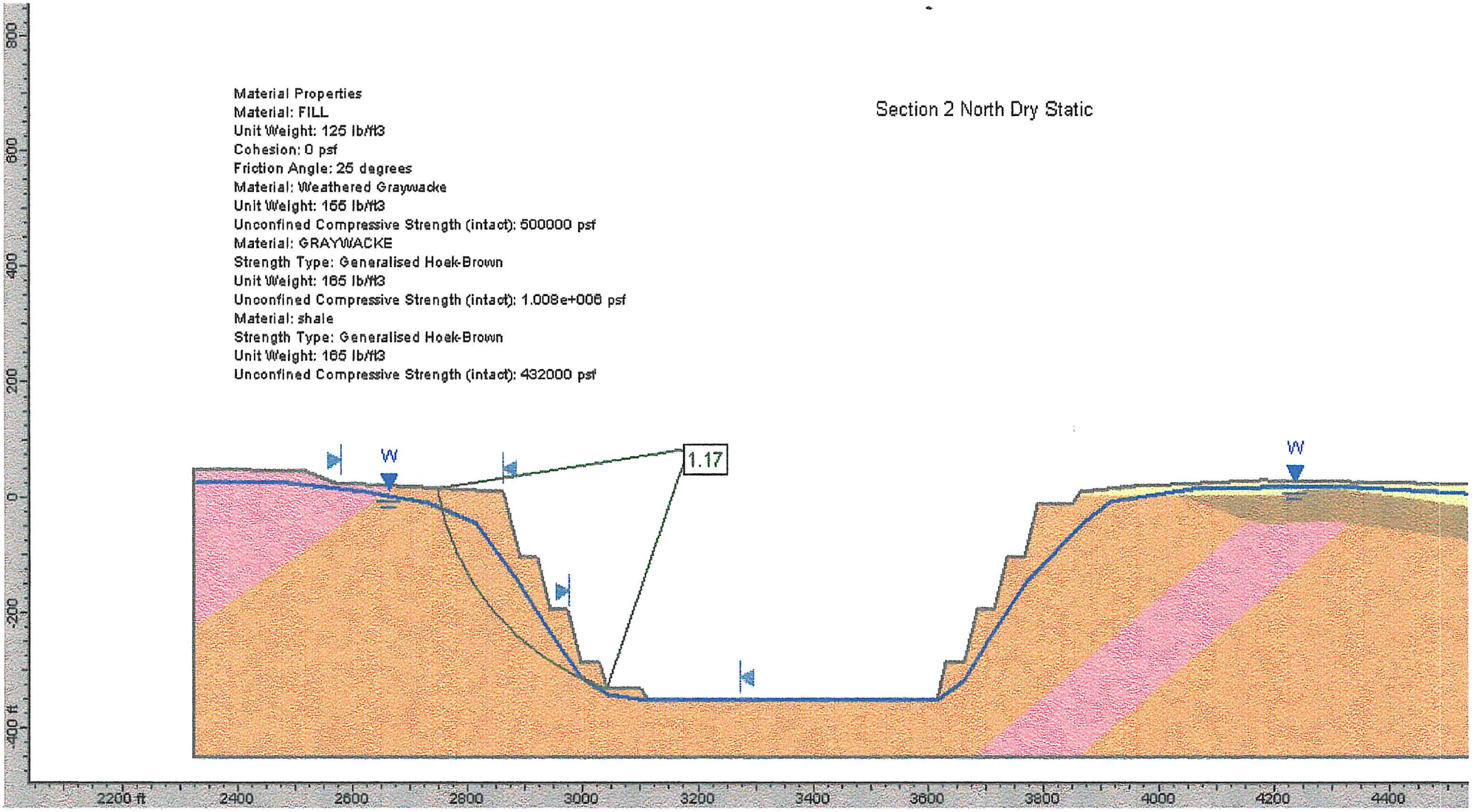
Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (Intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (Intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (Intact): 432000 psf
mb: 0.49251
s: 0.000418942
a: 0.522344
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3

Section 2 Existing, ~~South~~ North



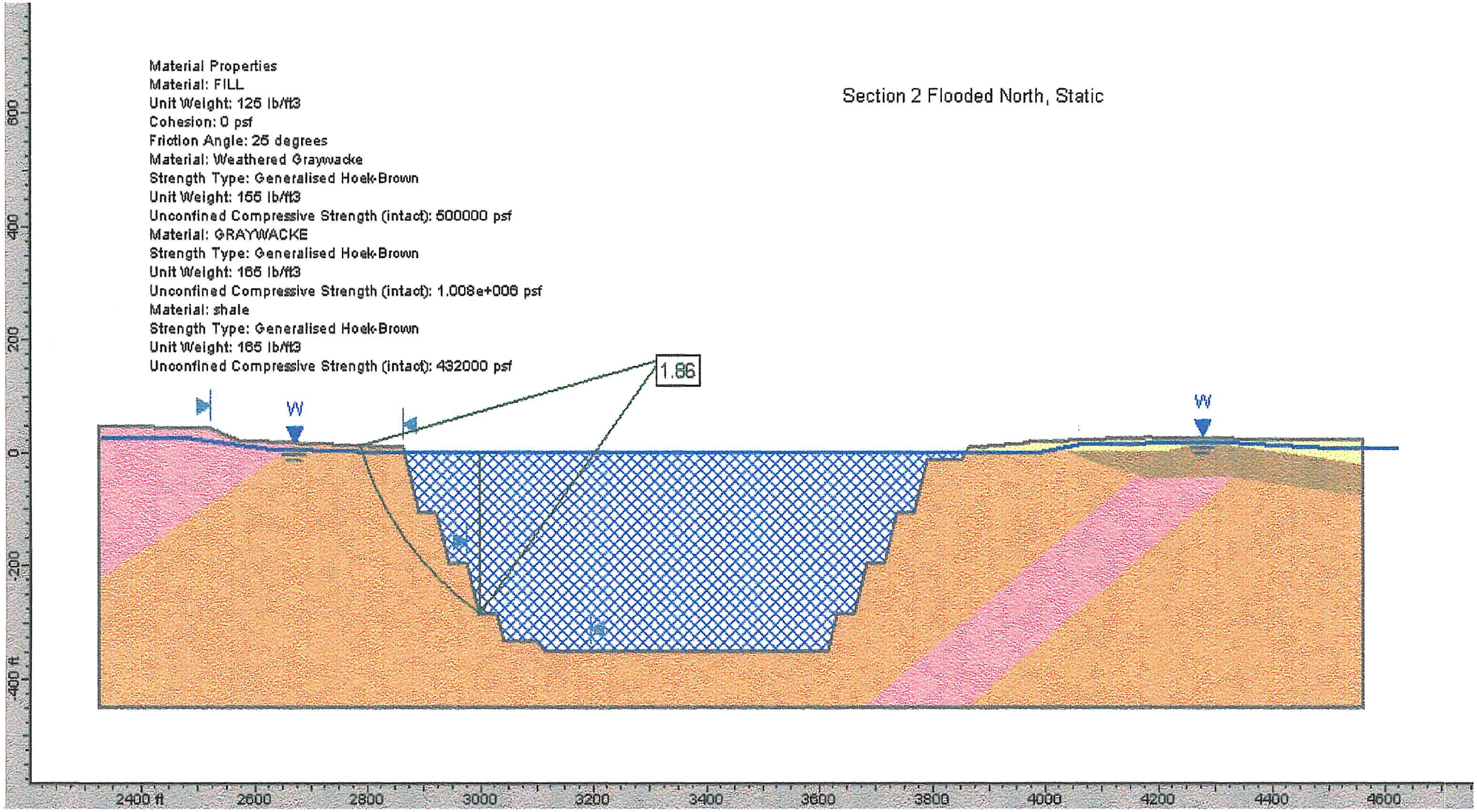
Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf

Section 2 North Dry Static



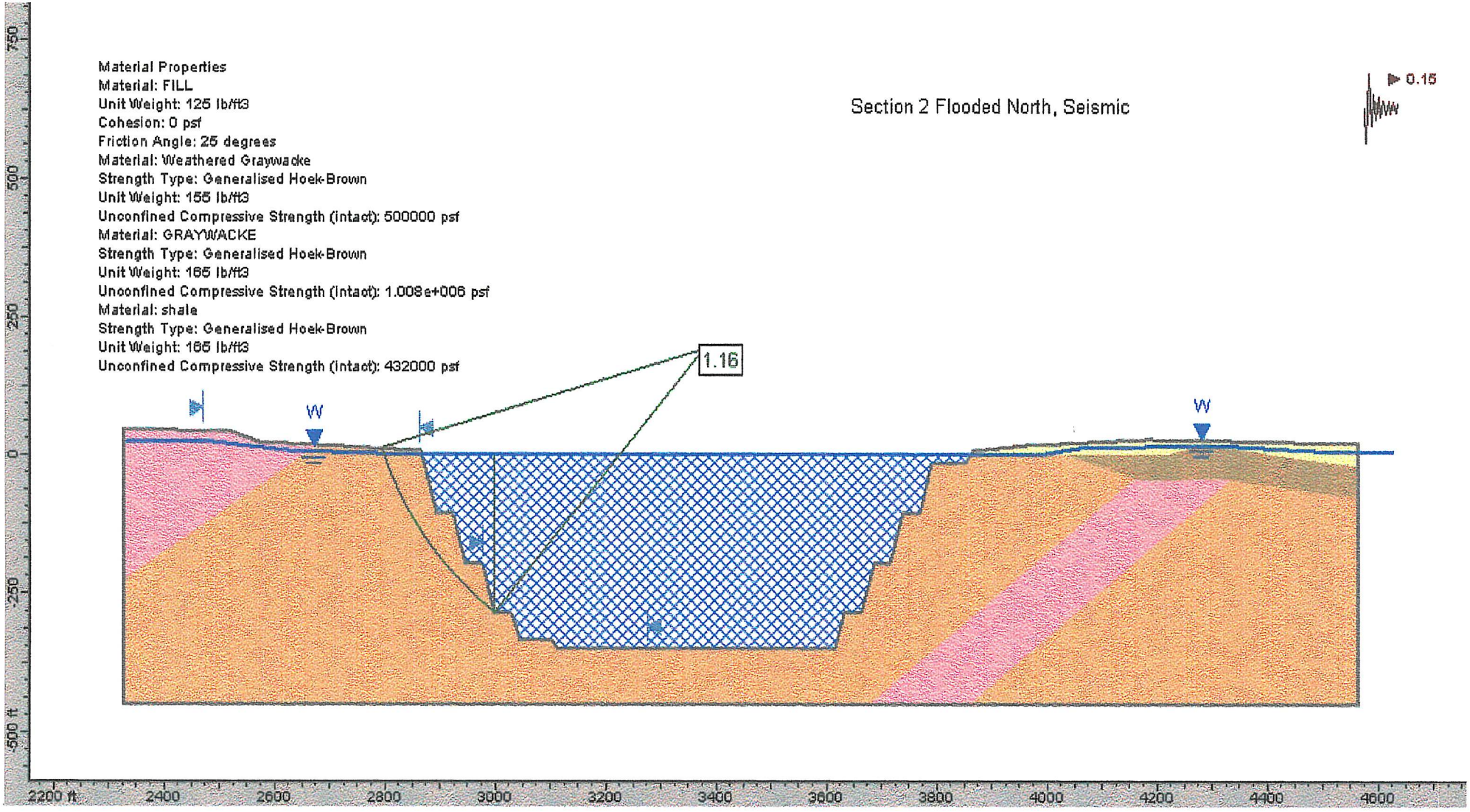
Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf

Section 2 Flooded North, Static



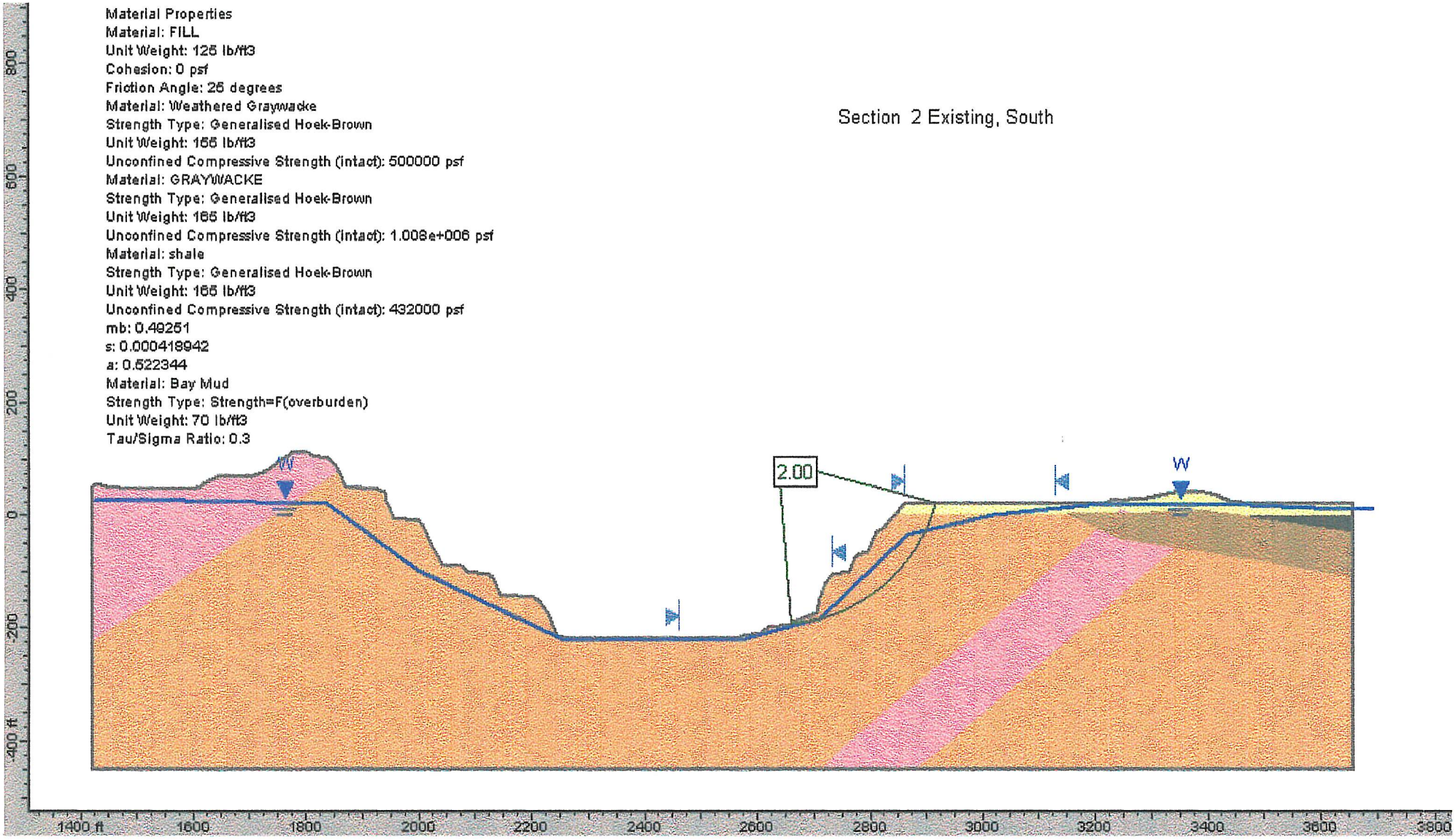
Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf

Section 2 Flooded North, Seismic



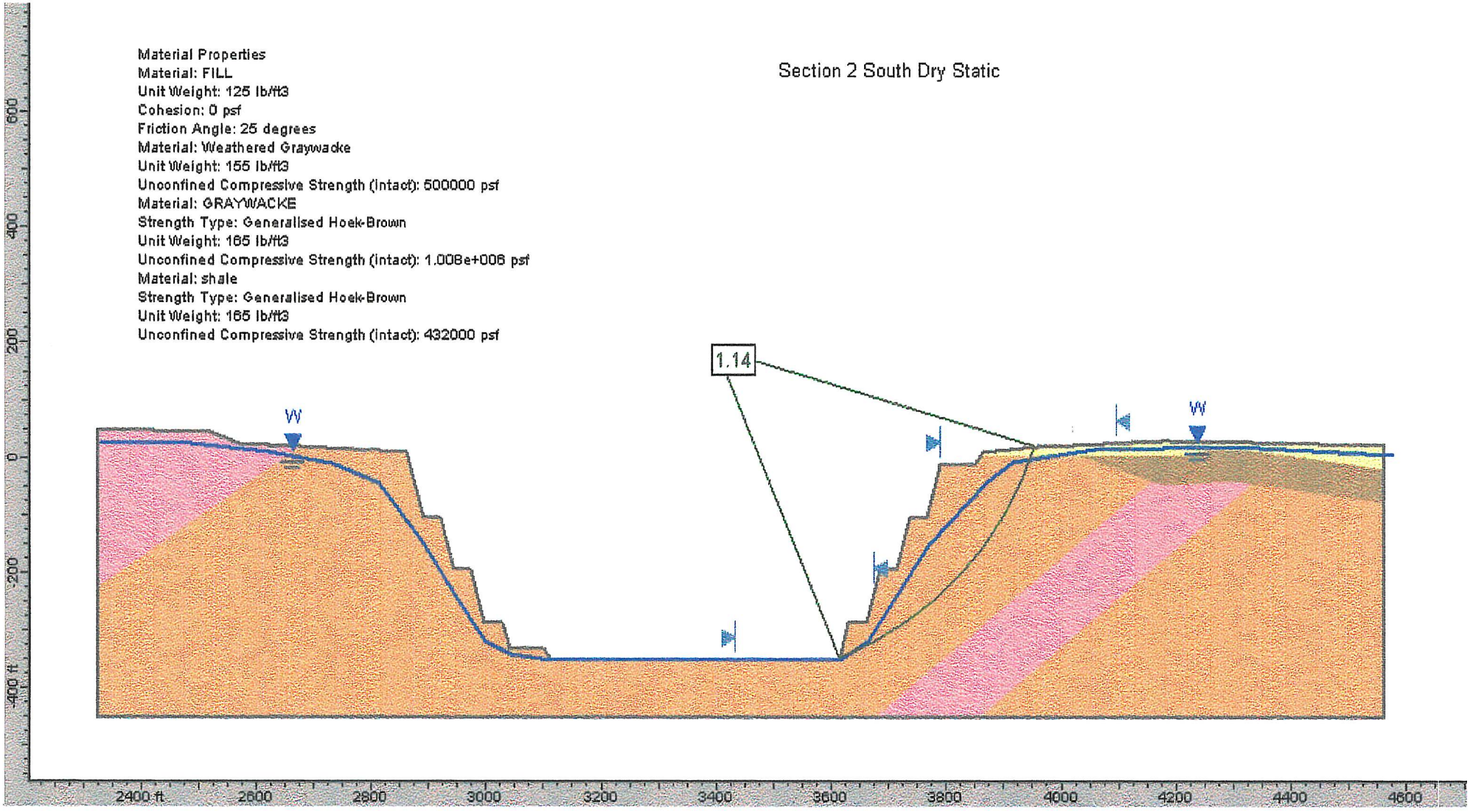
Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf
mb: 0.49251
s: 0.000418942
a: 0.522344
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3

Section 2 Existing, South



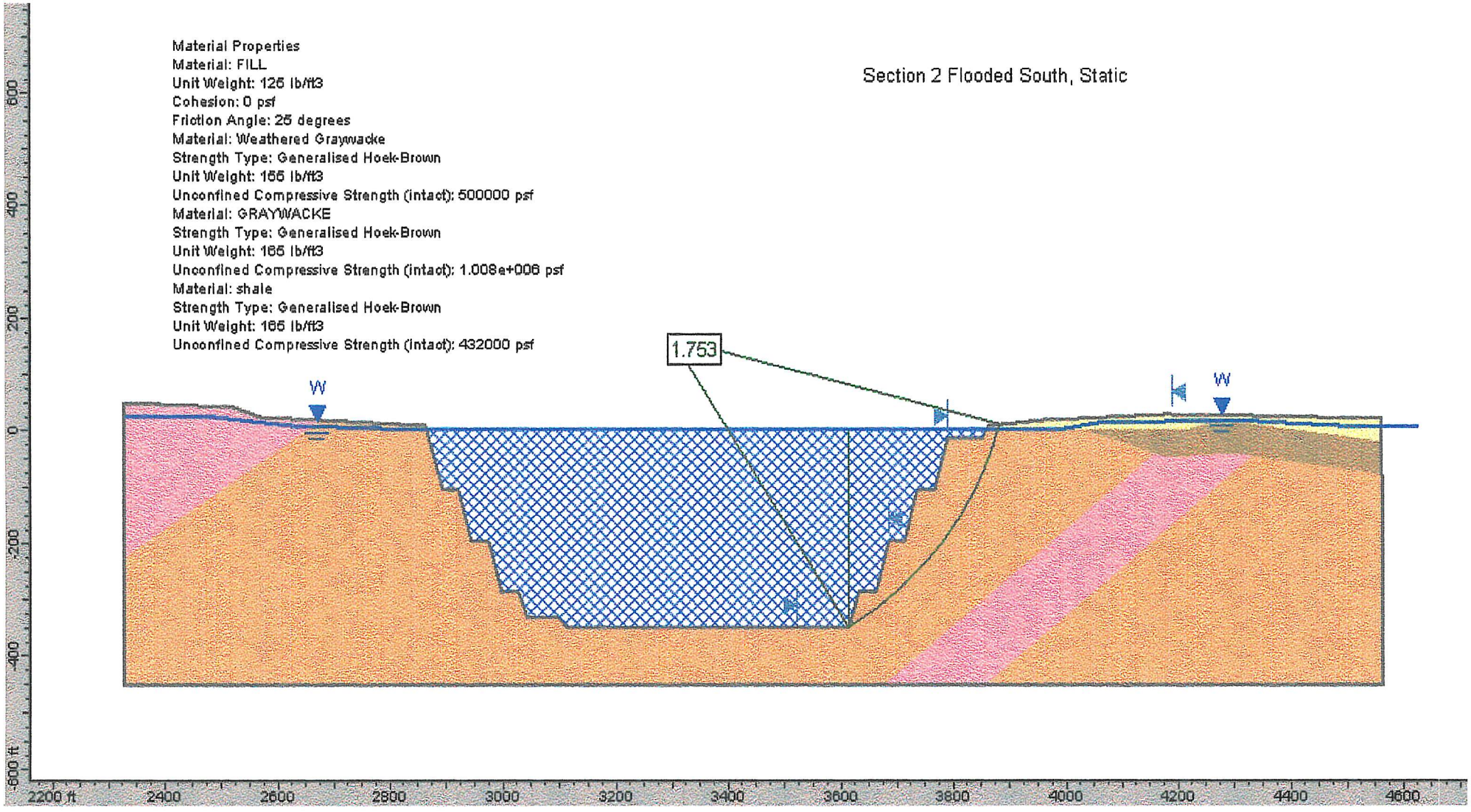
Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf

Section 2 South Dry Static



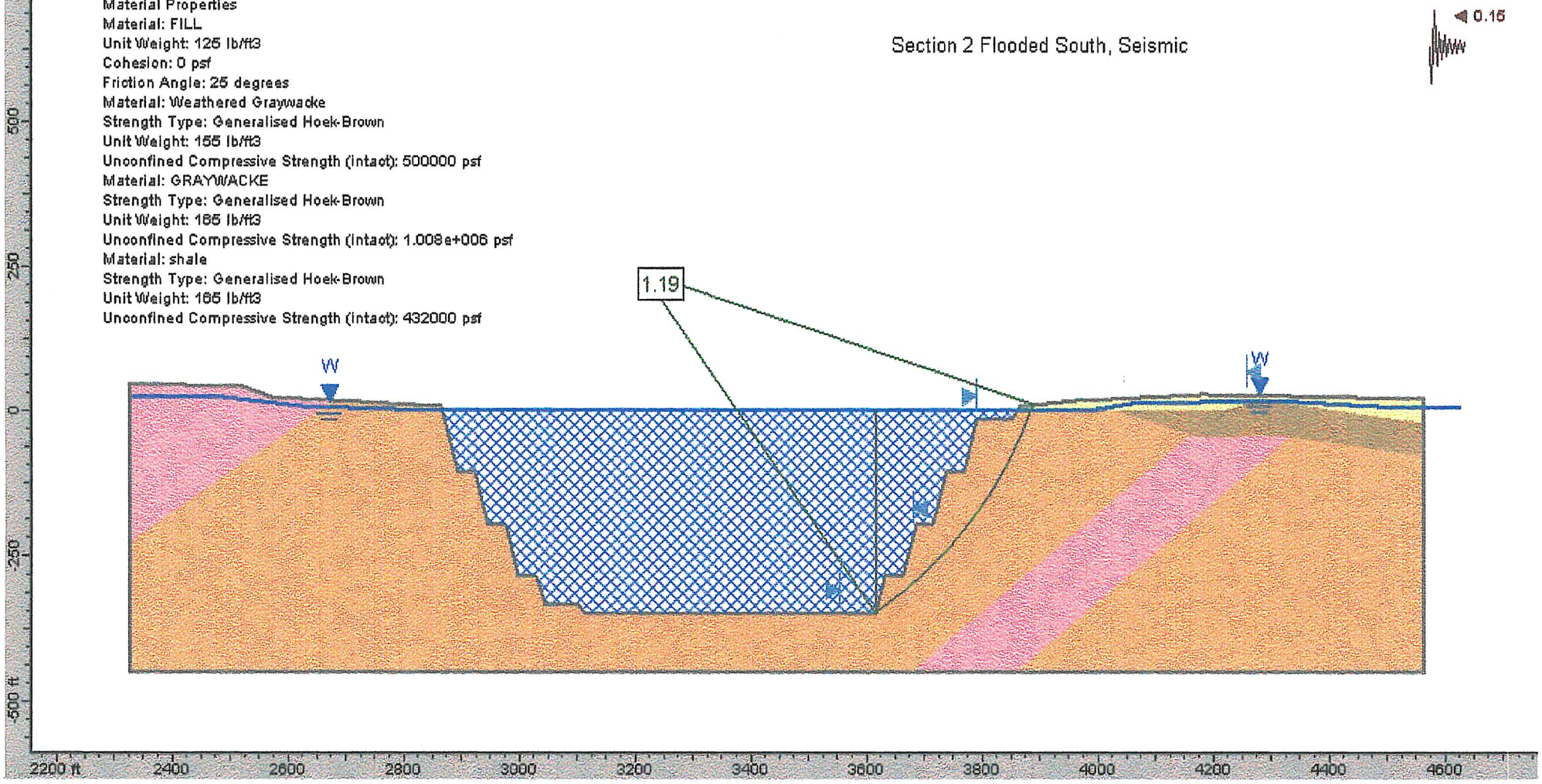
Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf

Section 2 Flooded South, Static



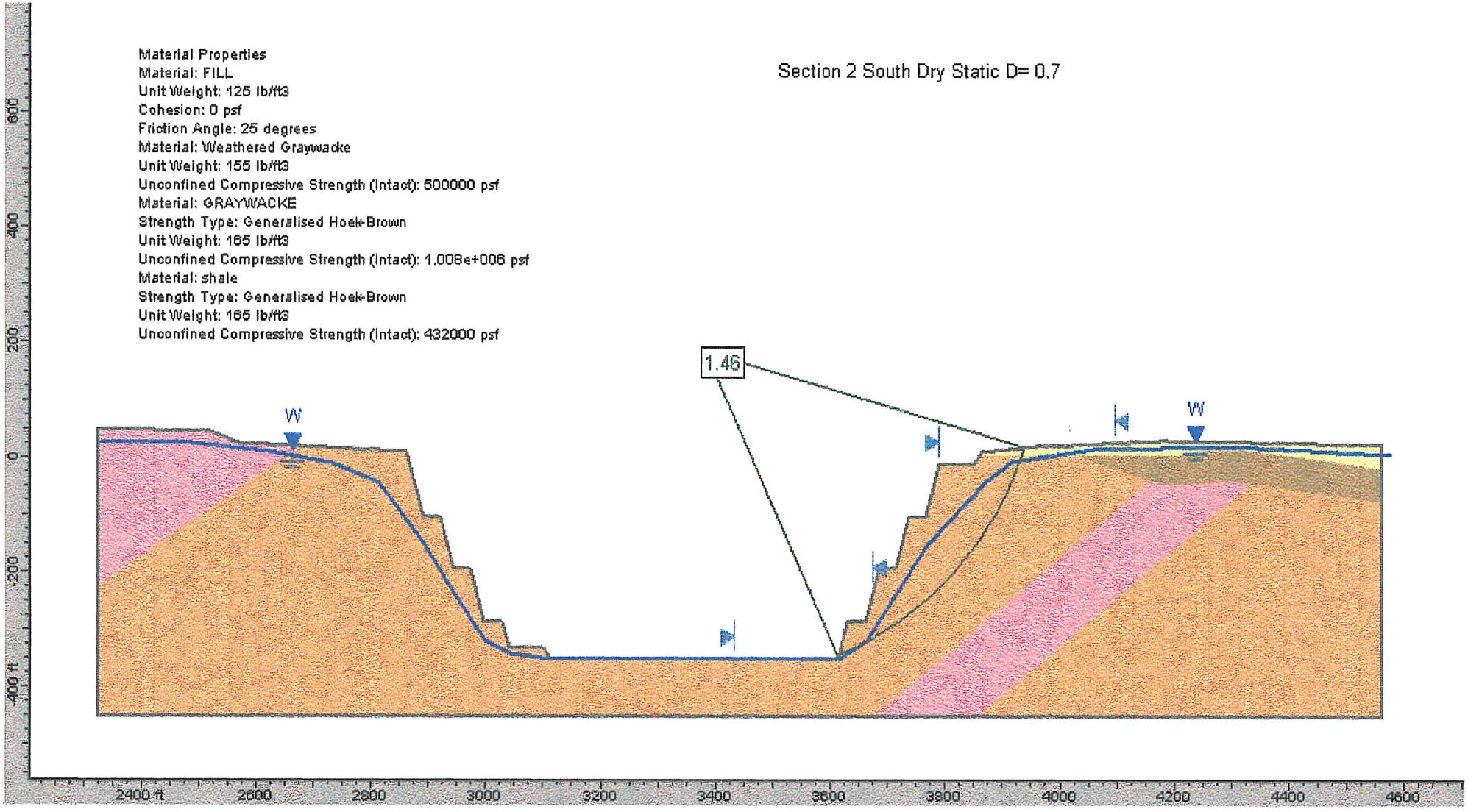
Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf

Section 2 Flooded South, Seismic



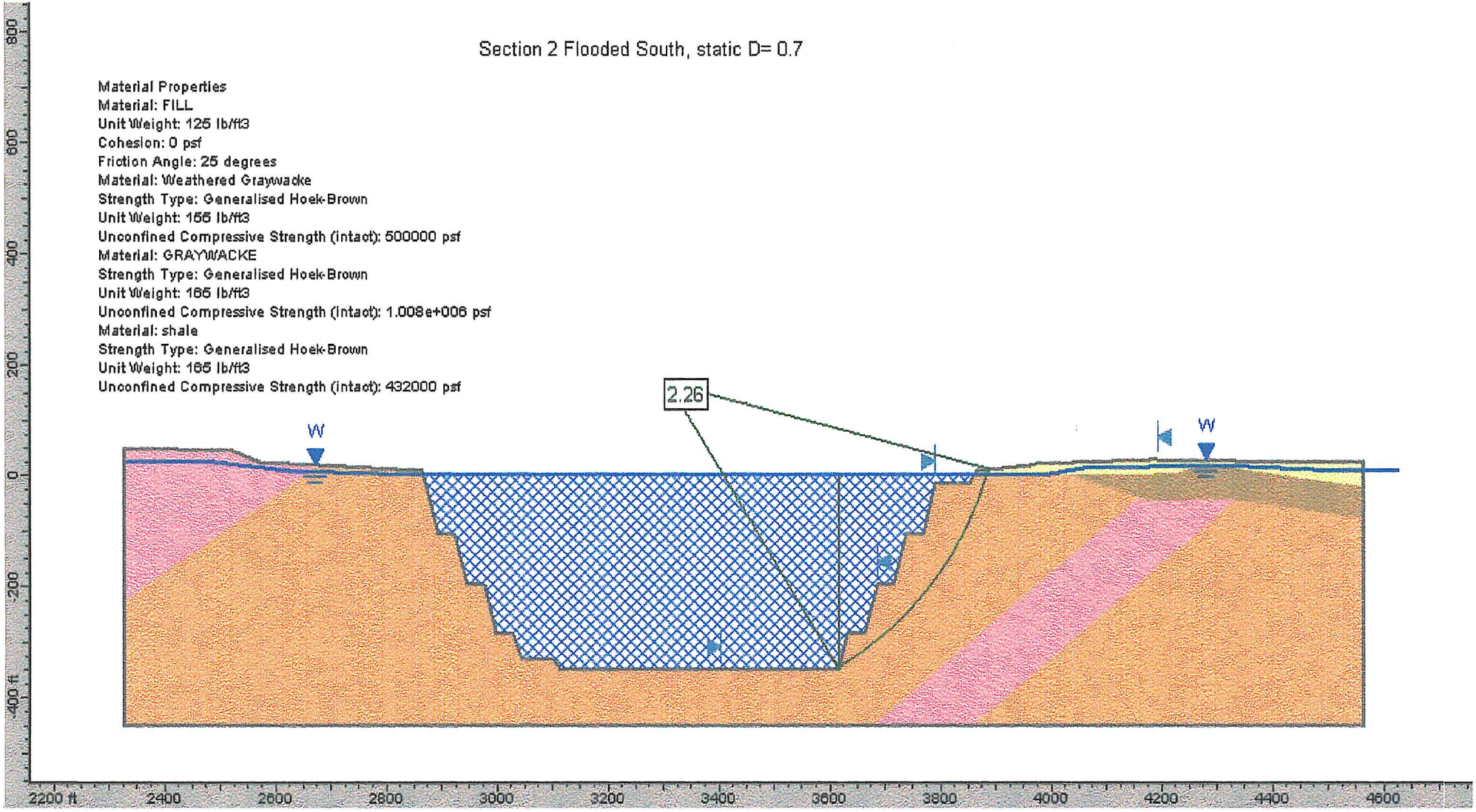
Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf

Section 2 South Dry Static D= 0.7



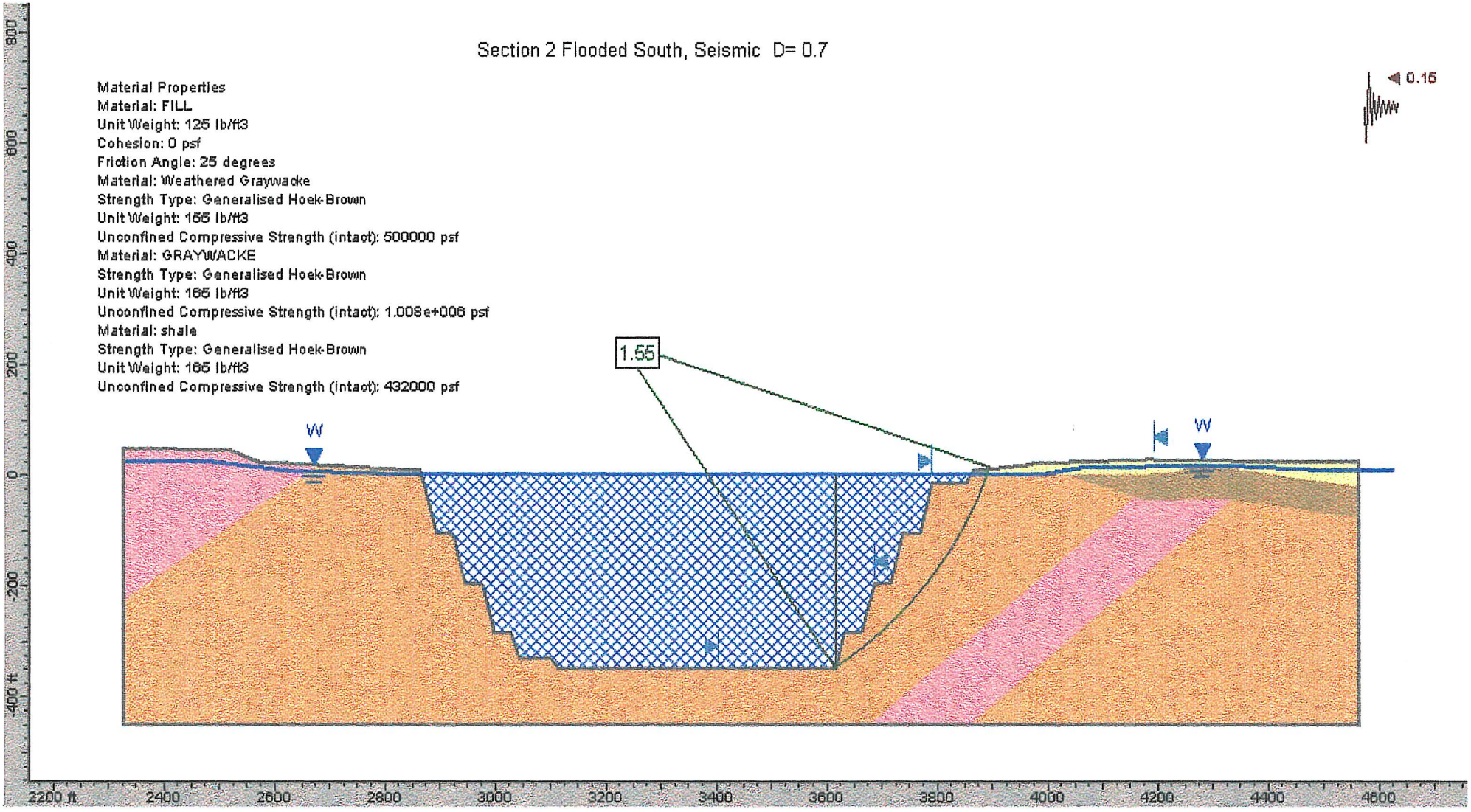
Section 2 Flooded South, static D= 0.7

Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf



Section 2 Flooded South, Seismic D= 0.7

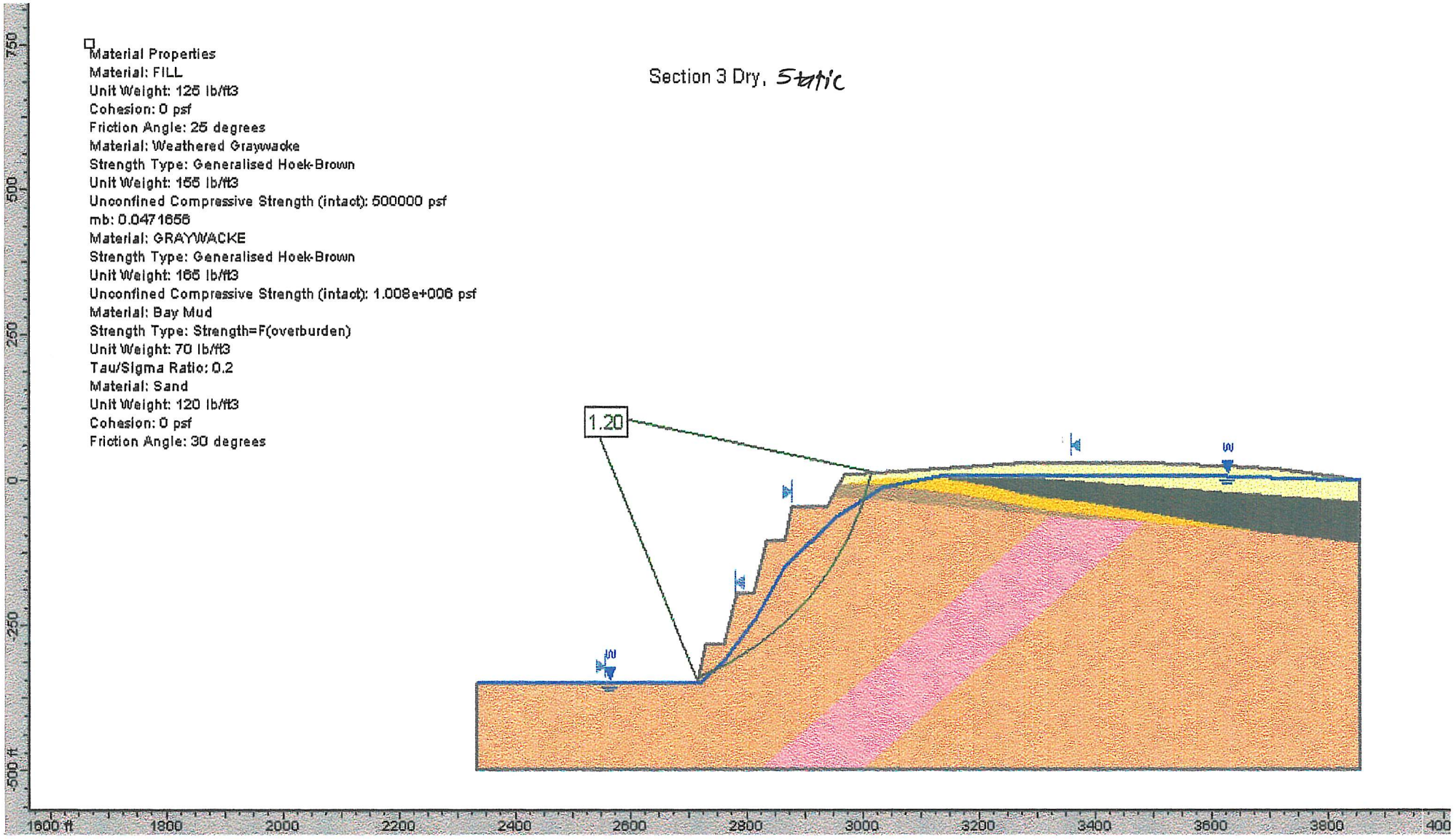
Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf



Cross Section 3

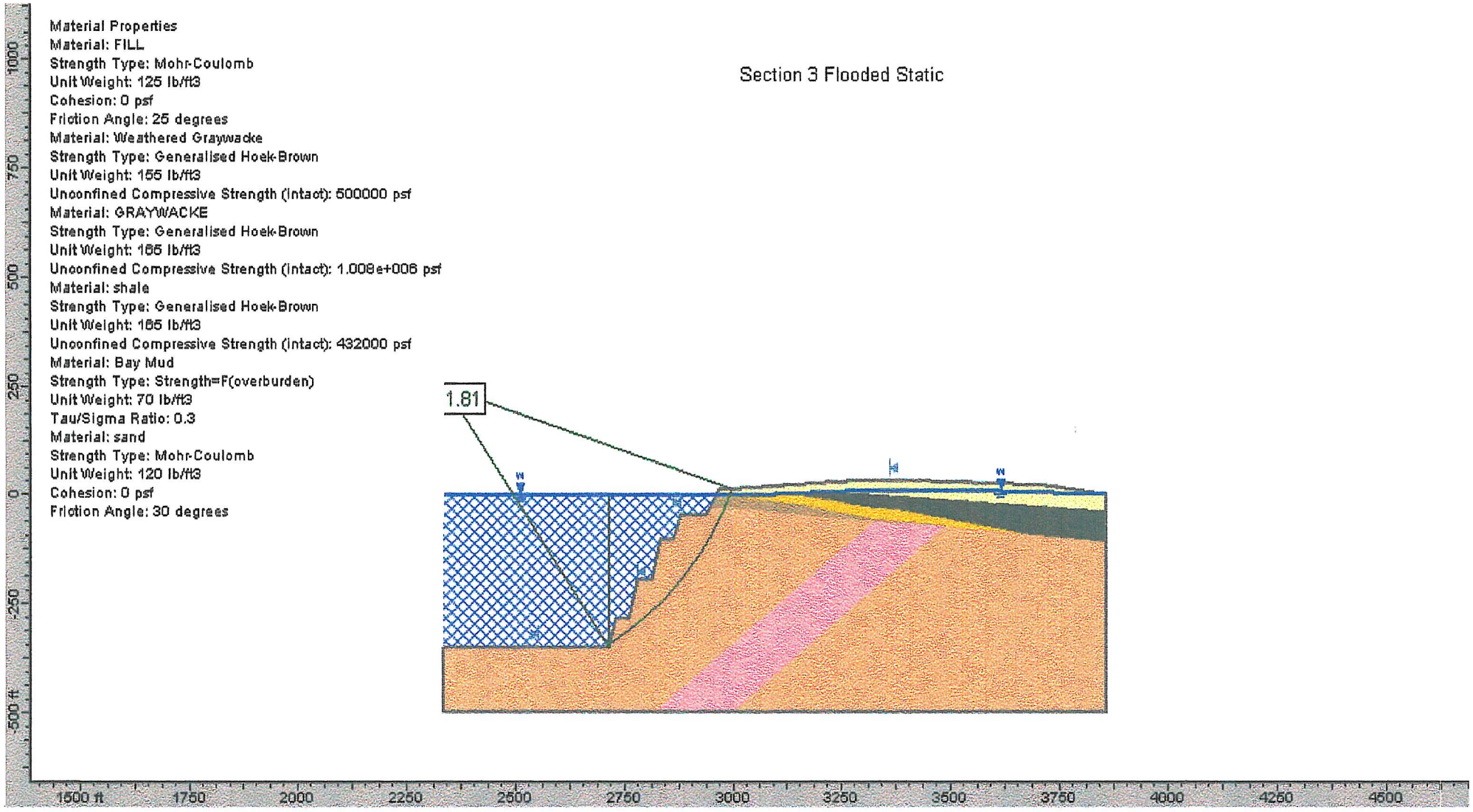
Section 3 Dry, Static

Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
mb: 0.0471656
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.2
Material: Sand
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees



Material Properties
Material: FILL
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3
Material: sand
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees

Section 3 Flooded Static



Material Properties
Material: FILL
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
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Strength Type: Generalised Hoek-Brown
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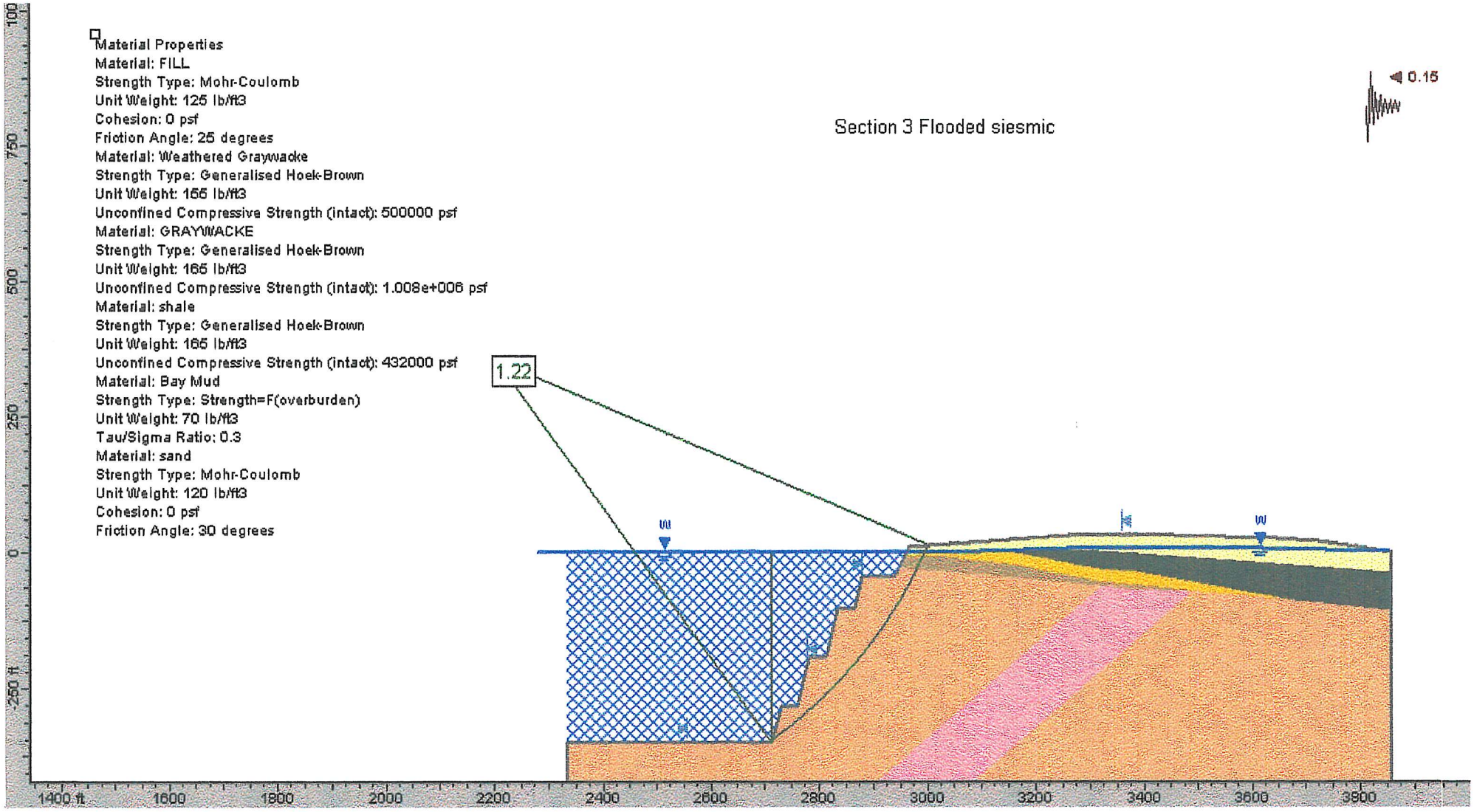
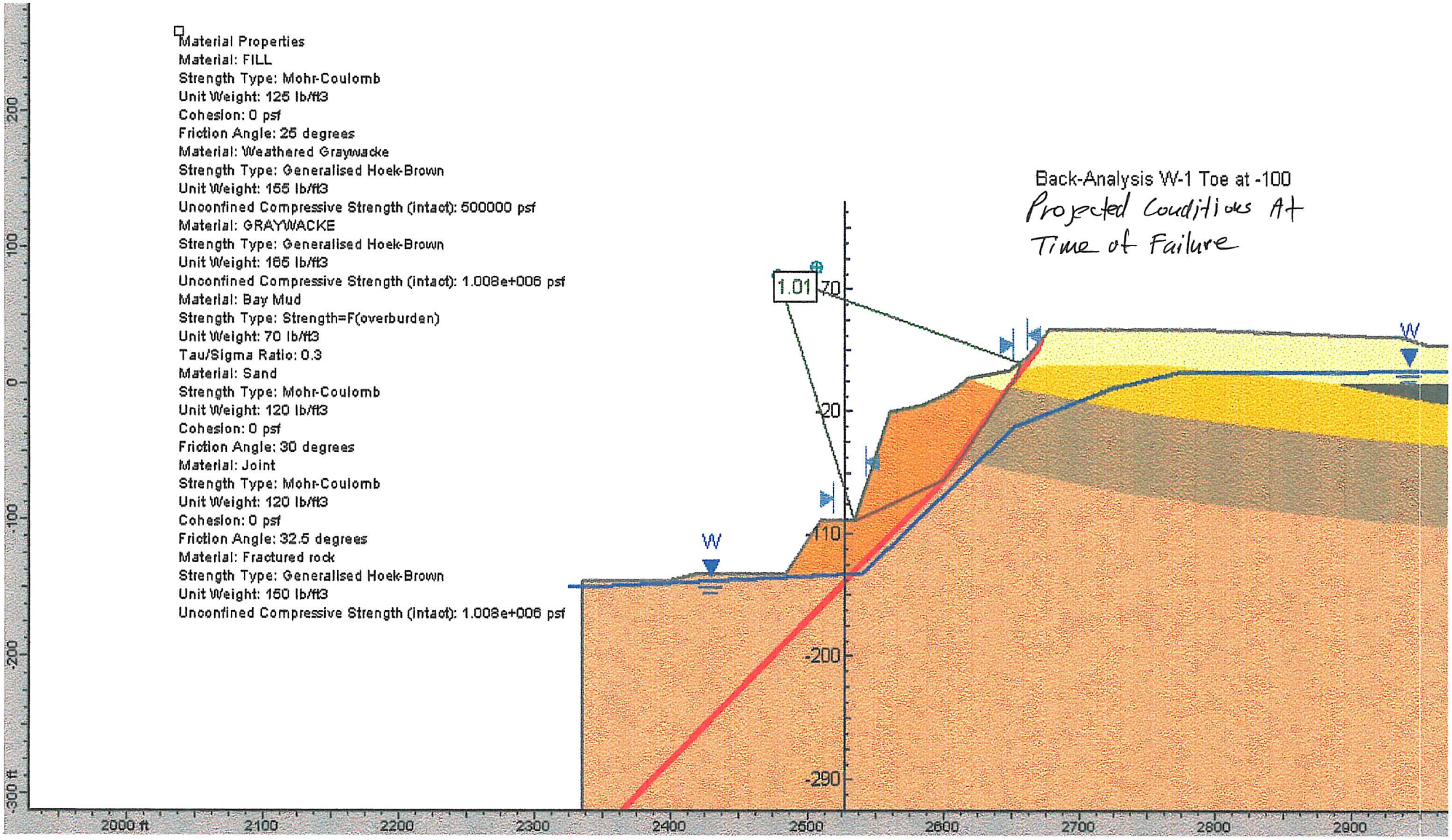
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3
Material: sand
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees

Section 3 Flooded siesmic



Cross Section 4

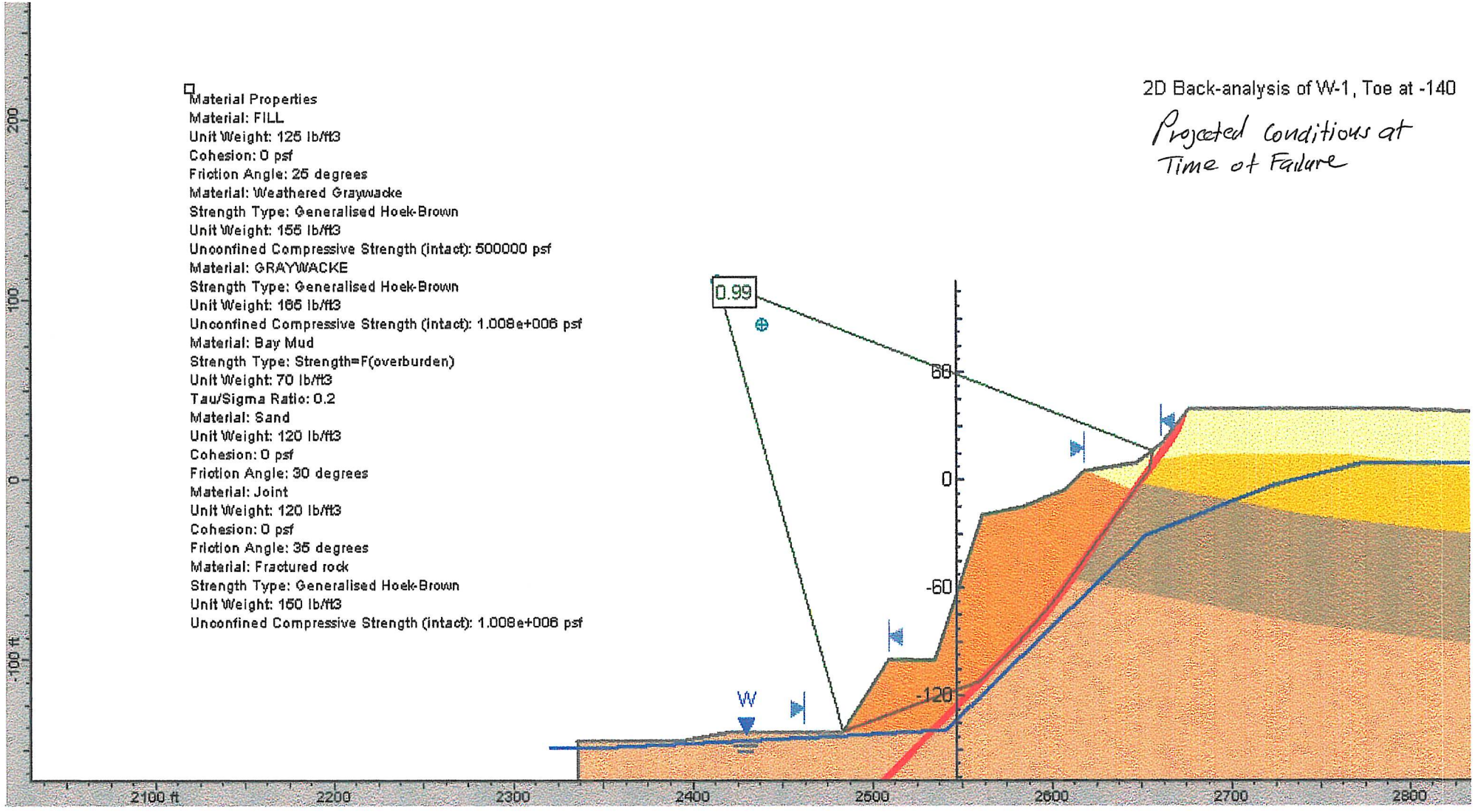
Material Properties
Material: FILL
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.3
Material: Sand
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees
Material: Joint
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 32.5 degrees
Material: Fractured rock
Strength Type: Generalised Hoek-Brown
Unit Weight: 150 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf



2D Back-analysis of W-1, Toe at -140

Projected Conditions at Time of Failure

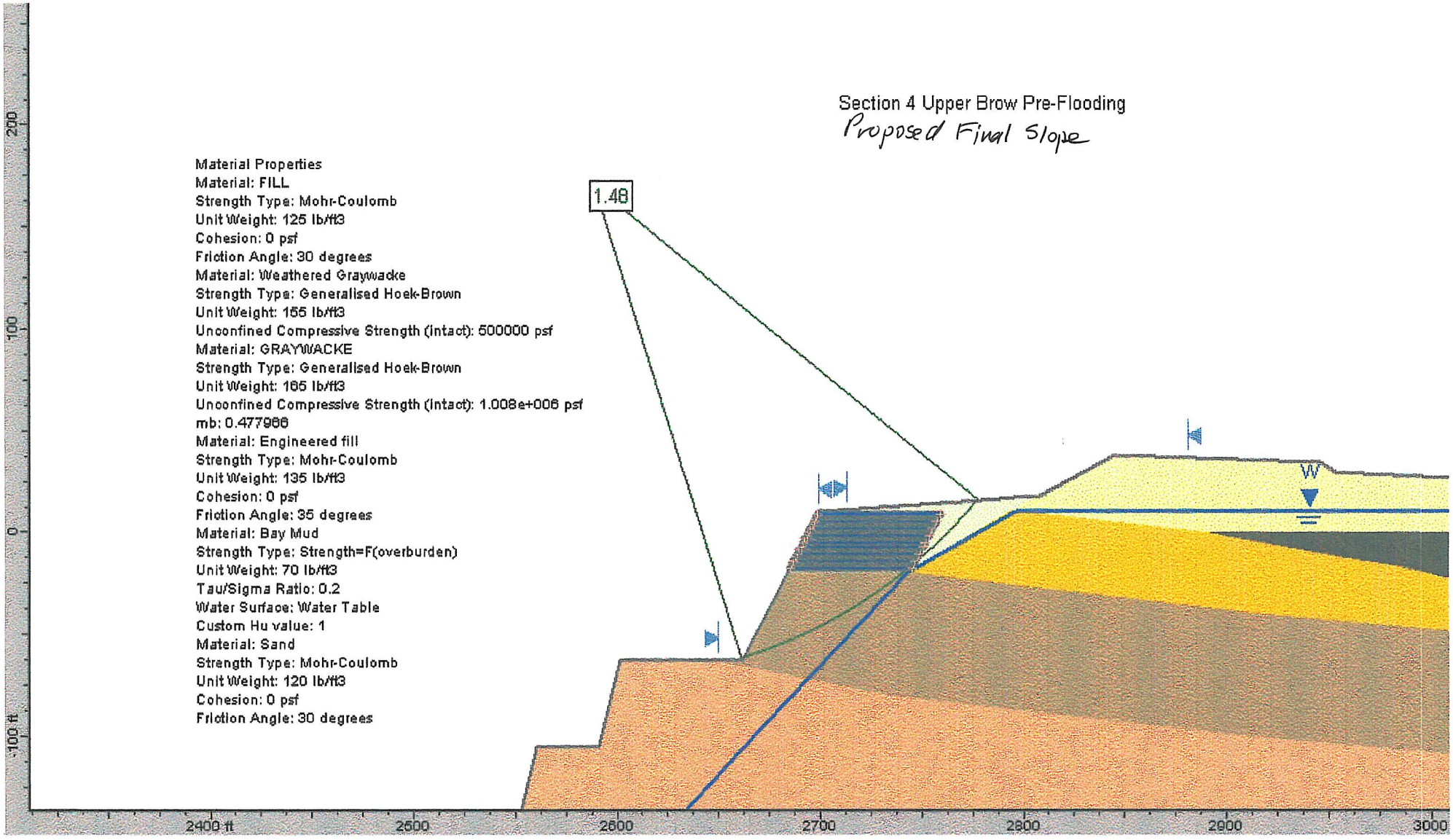
Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.2
Material: Sand
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees
Material: Joint
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 35 degrees
Material: Fractured rock
Strength Type: Generalised Hoek-Brown
Unit Weight: 150 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf



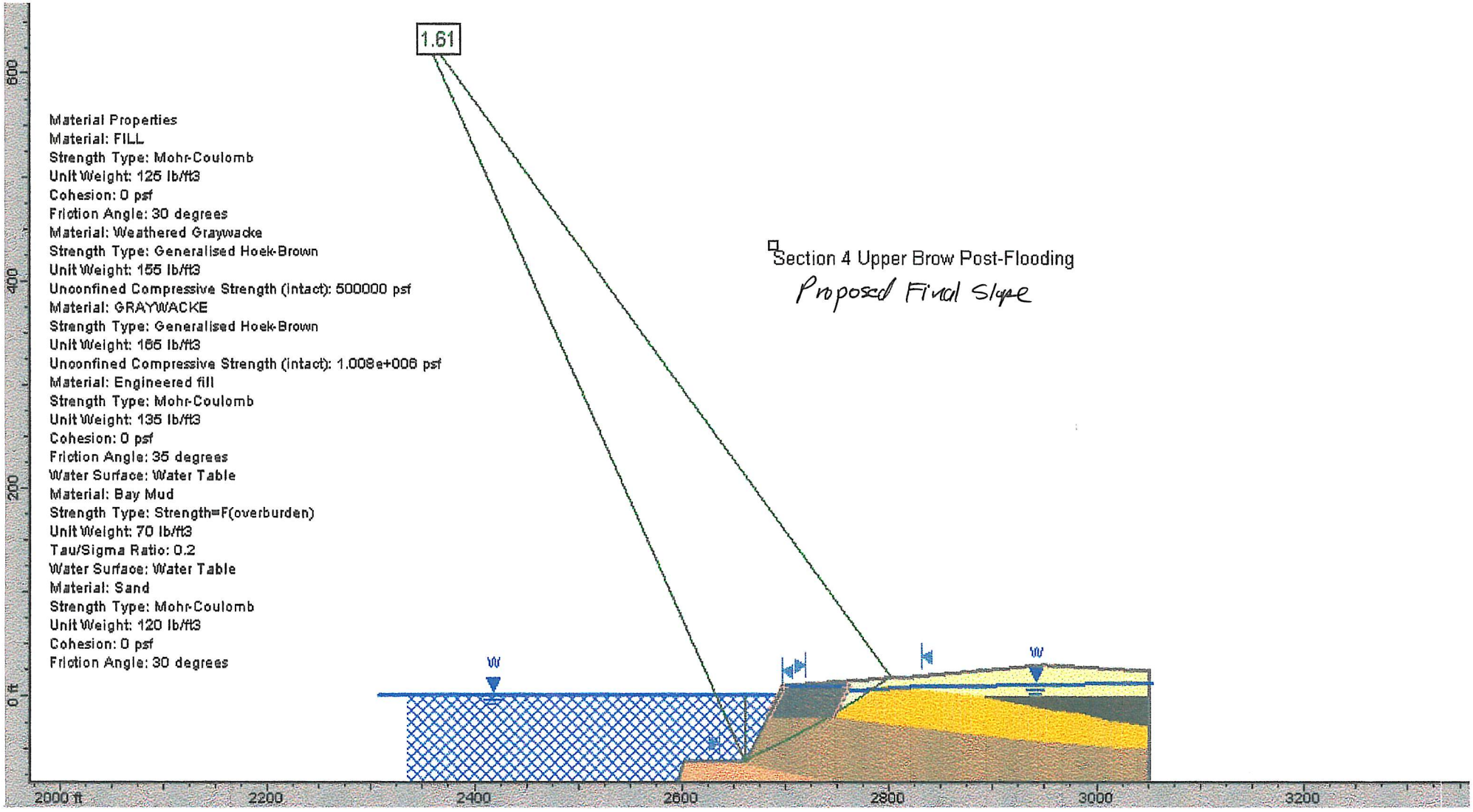
Section 4 Upper Brow Pre-Flooding

Proposed Final Slope

Material Properties
Material: FILL
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (Intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (Intact): 1.008e+006 psf
mb: 0.477966
Material: Engineered fill
Strength Type: Mohr-Coulomb
Unit Weight: 135 lb/ft3
Cohesion: 0 psf
Friction Angle: 35 degrees
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.2
Water Surface: Water Table
Custom Hu value: 1
Material: Sand
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees



Material Properties
Material: FILL
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: Engineered fill
Strength Type: Mohr-Coulomb
Unit Weight: 135 lb/ft3
Cohesion: 0 psf
Friction Angle: 35 degrees
Water Surface: Water Table
Material: Bay Mud
Strength Type: Strength=F(overburden)
Unit Weight: 70 lb/ft3
Tau/Sigma Ratio: 0.2
Water Surface: Water Table
Material: Sand
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees



Material Properties
Material: FILL
Strength Type: Mohr-Coulomb
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
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Material: Engineered fill
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Unit Weight: 135 lb/ft3
Cohesion: 0 psf
Friction Angle: 35 degrees
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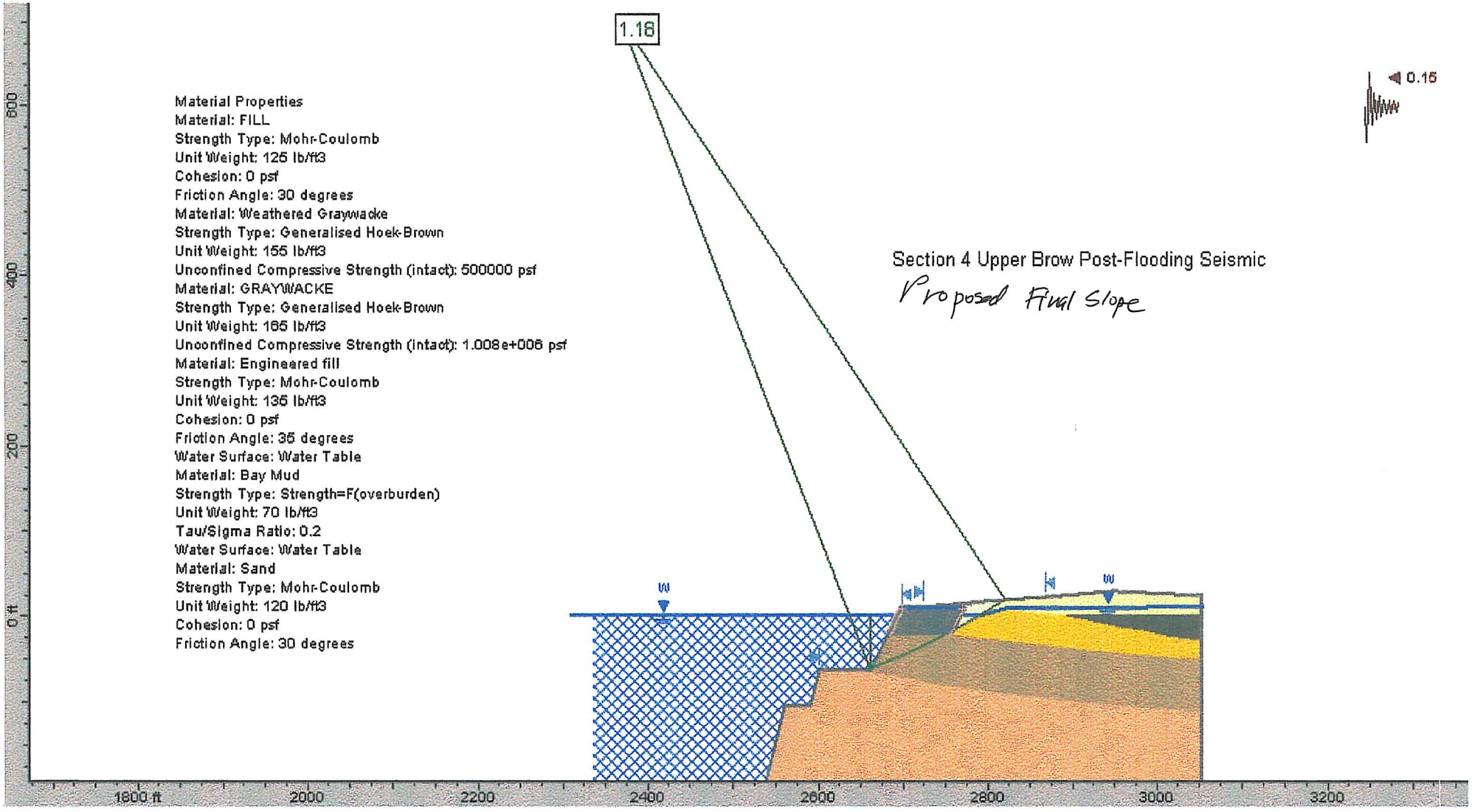
Material: Sand
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 0 psf
Friction Angle: 30 degrees



Seismic Displacement Analyses

6261.1.003.01
April 11, 2005

NEWMARK DISPLACEMENT ANALYSIS

Project Name: San Rafael Quarry
Project Number: 6261.1.003.01
Engineer: Jeffrey Wisniewski
Evaluation: Section 2, Flooded South
Procedure: Makdisi & Seed, 1978

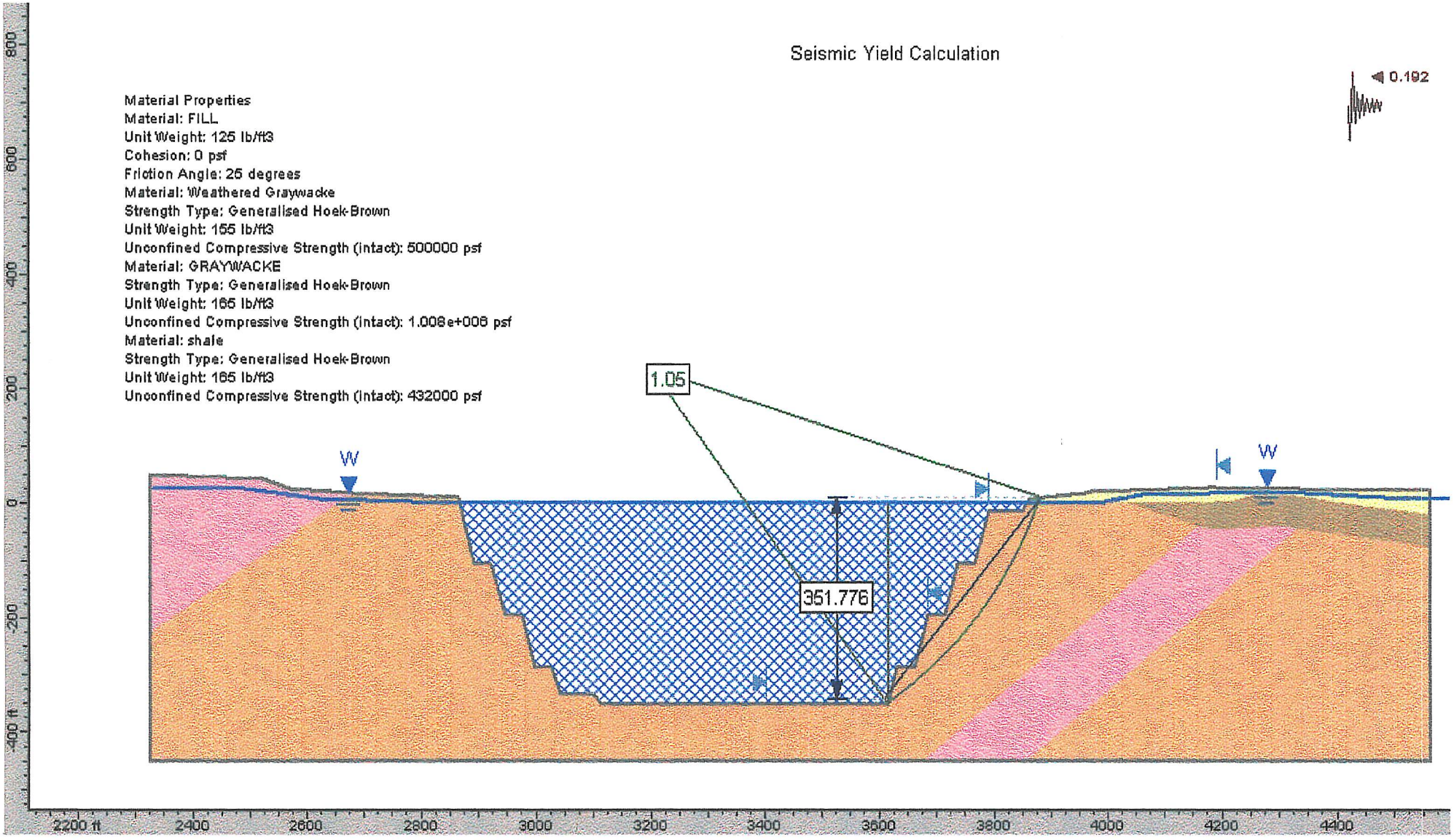
SIMPLIFIED PROCEDURE

Peak Ground Acceleration, PGA_{rock}	0.6	g	
Slope Height, H	350	ft	
Depth of Failure Wedge, h	300	ft	
Depth to Height Ratio, h/H	0.86		
Ratio, k_{max}/PGA_{max}	0.380		(Figure 7)
Max Seismic Acceleration, $k_{max} = (k_{max}/PGA_{rock}) \cdot PG$	0.228	g	
Yield Acceleration, k_y	0.192	g	(FROM SLOPE STABILITY ANALYSIS, MINIMUM SEISMIC ACCELERATION NECESSARY FOR FS=1)
k_y/k_{max}	0.842		
Displacement, U	0.2-0.9	cm	(Figure 9b)

Deformations unlikely to result from earthquake event. (SP 117)

Seismic Yield Calculation

Material Properties
Material: FILL
Unit Weight: 125 lb/ft3
Cohesion: 0 psf
Friction Angle: 25 degrees
Material: Weathered Graywacke
Strength Type: Generalised Hoek-Brown
Unit Weight: 155 lb/ft3
Unconfined Compressive Strength (intact): 500000 psf
Material: GRAYWACKE
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 1.008e+006 psf
Material: shale
Strength Type: Generalised Hoek-Brown
Unit Weight: 165 lb/ft3
Unconfined Compressive Strength (intact): 432000 psf



Makdisi and Seed (1978) Simplified Procedure for Estimating Embankment Earthquake-Induced Deformations

Calculational Procedure:

1. Evaluate material's strength loss potential

   - if significant, do not use this method

   - if slight, use reduced material shear strength (in most cases, a reduction < 20% of the static undrained strength is reasonable)

2. Calculate the yield acceleration, k_y, as the seismic coefficient that produces a FS = 1.0 in pseudo-static slope stability analyses

3. Estimate the maximum crest acceleration, $\ddot{u}_{max}$ and first natural period, T_o, due to a specified earthquake

   - calculate $\ddot{u}_{max}$ and T_o with 1-D or 2-D dynamic analyses (e.g., SHAKE91, Idriss and Sun, 1992; FLUSH, Lysmer et al., 1975)

   - estimate $\ddot{u}_{max}$ and T_o from observations (e.g., Harder, 1991, Boulanger et al., 1993)

   - use approximations:

$$1\text{-D},\ T_o \approx \frac{4H}{V_s}$$

$$2\text{-D},\ T_o \approx \frac{2.61H}{V_s}$$

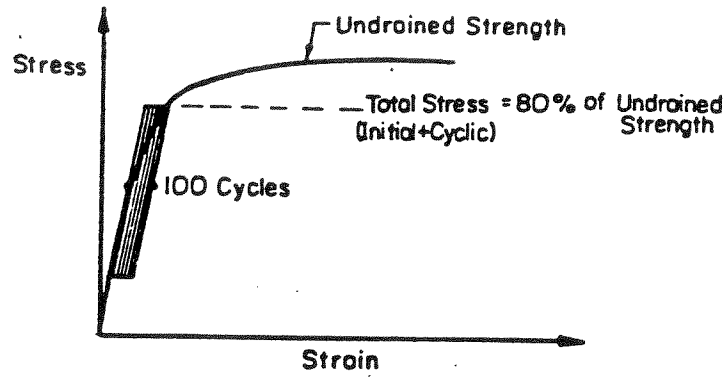
Ambrayseys and Sarma (1967)

4. Using plot of maximum acceleration ratio ($k_{max}/\ddot{u}_{max}$) vs. depth of sliding mass (y/h), estimate k_{max} for specified sliding mass

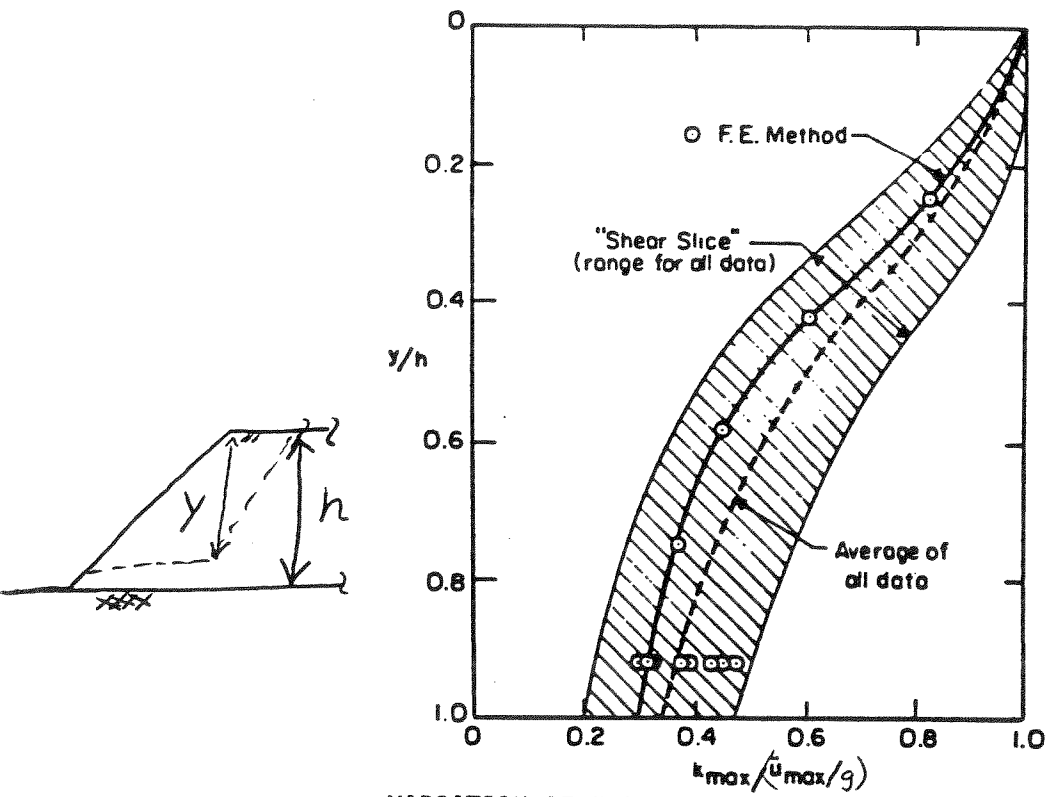
5. Enter curves of average normalized permanent displacement ($u/k_{max}\ g\ T_o$) vs. yield acceleration ratio (k_y/k_{max}) to estimate permanent displacement, U.

Note: (1) In lieu of step 5, estimate permanent displacement, U, directly from plot of U vs. k_y/k_{max} without estimate of T_o

(2) "Rigid" body sliding along failure plane only; for volumetric component within sliding mass, use Tokimatsu & Seed (87)



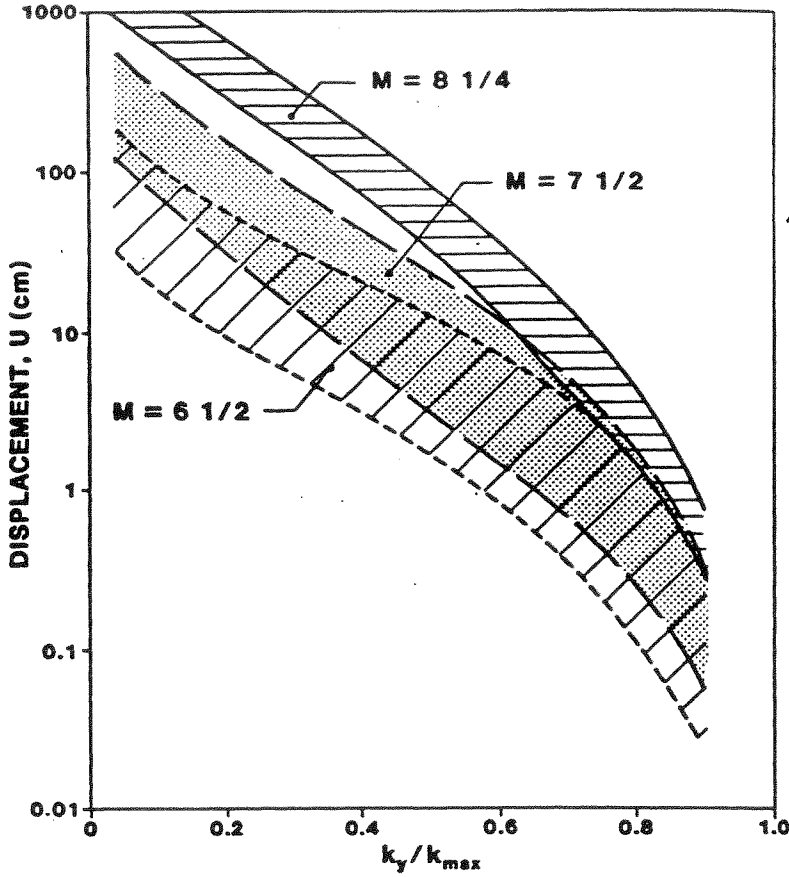
DEFINITION OF DYNAMIC YIELD STRENGTH



$K_{max} \approx MHEA/g$

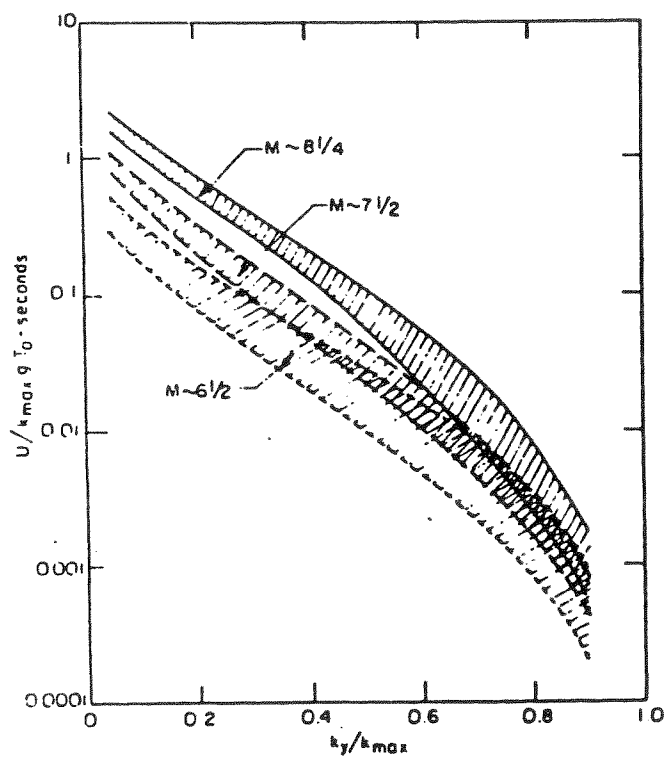
VARIATION OF "MAXIMUM ACCELERATION RATIO" WITH DEPTH OF SLIDING MASS

Makdisi, F.I., and Seed, H.B. (1978)



USE THIS PLOT & NOT ONE IN Makdisi & Seed (1978) ASCE Pub. (It contains a printing error)

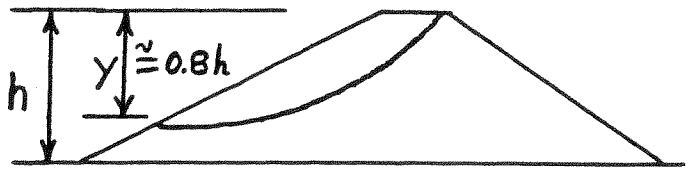
VARIATION OF PERMANENT DISPLACEMENT WITH YIELD ACCELERATION



VARIATION OF NORMALIZED PERMANENT DISPLACEMENT WITH YIELD ACCELERATION - SUMMARY OF ALL DATA

Makdisi, F.I., and Seed, H.B. (1978)

EXAMPLE



$\ddot{u}_{max}$ = 0.6 g

M = 7½

U = ?

<u>Solution</u>

1. Material does not lose significant strength; use Dynamic Strength, $S = 0.90\ S_u$

2. Using pseudo-static limit equilibrium analysis

k	0	0.1	0.15	0.14
FS	1.5	1.15	0.95	1.0

$\therefore\ k_y = 0.14$

3. $\ddot{u}_{max} = 0.6$ g $\qquad$ T_o is unknown

4. From Makdisi and Seed (1978) Figure 7, $\dfrac{k_{max}}{(\ddot{u}_{max}/g)} \approx 0.4$, given $y/h = 0.8$

$\therefore\ k_{max} \approx 0.24$ (i.e. 0.4 * 0.6)

5. $k_y/k_{max} = 0.14/0.24 = 0.6$

From Makdisi and Seed (1978) Figure 9(b), U ≈ 1 - 10 cm
.03 -.3 ft

Seismic Coefficient Calculations

6261.1.003.01
April 11, 2005

DEVELOP SEISMIC COEFFICIENT FOR STEEP SLOPES FOR PSEUDO-STATIC SLOPE STABILITY (ASHFORD & STAR, 2002)

(1) PGA = 0/60g

(2) Acceleration @ Crest

a_{max} = 1.5 PGA (Ashford & Sitar, 1997)
a_{max} = 1.5 (0.60g) = 0.90g

(3) h/H = 1.0
k_{max} = 0.325 (Makdisi & Seed, 1978)
k_{max} = 0.325 x 0.90g ≈ 0.2925g

(4) Average Maximum Seismic Coefficient

K_{au} = 0.65 K_{max} (Seed & Martin, 1966)
K_{au} = 0.65 x 0.2925g ≈ 10.190g

(5) Use Seismic Coefficient of 0.190g for pseudo static analyses