APPENDICES

A) GEOTECHNICAL CONSTRAINTS MEMORANDUM (ENGEO, INC.)

Project No.
5687.100.104

January 16, 2013
Revised January 30, 2013

Mr. Angelo Obertello
Carlson Barbee & Gibson
6111 Bollinger Canyon Road, Suite 150
San Ramon, CA 94583

Subject: Alameda Point – Infrastructure Planning
Alameda, California

GEOTECHNICAL CONSTRAINTS

References:

1. A3GEO, Inc. and Alan Kropp & Associates, Inc.; Data Report, Preliminary Geotechnical and Geologic Studies, Lawrence Berkeley National Laboratory Future Scientific Facility, Alameda Point, Alameda, California; October 28, 2011.

2. ENGEO; Preliminary Geotechnical Exploration, Alameda Point Development, Alameda, California; April 8, 2003; Project No. 5497.100.102.

3. Subsurface Consultants Inc.; Geotechnical Investigation, Oakland Harbor Navigation Improvement (-50 foot) Project, Port of Oakland, Oakland and Alameda, California; February 12, 1999.

4. Carlson, Barbee & Gibson Inc.; Alameda Point, Master Infrastructure Plan, Base Case – Reuse Plan, Land Use and Zoning Districts; October 11, 2012.

Dear Mr. Obertello:

At your request, we prepared the following discussion of the geotechnical constraints that will impact redevelopment of Alameda Point in Alameda, California. We understand that the City of Alameda (City) is advancing site development planning. The purpose of this study is to assist in infrastructure planning at the site. The referenced documents were utilized for this study:

SITE DESCRIPTION AND PROJECT DESCRIPTION

Alameda Point is an area located on the westerly portion of Alameda Island in the City of Alameda, California. Alameda Island lies along the eastern side of the San Francisco Bay, adjacent to the City of Oakland. The site is a portion of the former Naval Air Station Alameda that ceased operations as a military base in 1997. The site is roughly rectangular in shape and is approximately 2 miles long and 1 mile wide. Based on a planning document by Carlson, Barbee & Gibson Inc., (Reference 4), the City is currently interested in developing an infrastructure plan

in order to facilitate redevelopment of the site with a mixture of housing, commercial, retail, marine-related facilities, and open spaces.

PREVIOUS GEOTECHNICAL DOCUMENTS

Numerous previous geotechnical explorations have been performed at the site during history. Reports by Subsurface Consultants Incorporated in 1999, ENGEO in 2003, and A3GEO, Inc. and Alan Kropp & Associates, Inc. in 2011, References 1, 2, and 3, are highly relevant to the current study. Numerous borings, Cone Penetration Tests (CPTs) and lab tests were included in these studies. We have compiled and selectively used, as deemed appropriate, the previous field and laboratory data in this current study. The approximate locations of the previous explorations are illustrated on Figure 1 (Site Plan).

SUBSURFACE CONDITIONS

Based on our review of the subsurface information in References 1 through 3, artificial fill of varying thickness was encountered in historic explorations throughout the site. Young Bay Mud was encountered beneath the fill in the portions of the site to the north of the seaplane lagoon with the greatest thickness approximately 130 feet. Merritt Sand and the San Antonio formation sand were found directly beneath the fill in the southeastern portion of the site (approximately 60 to 70 feet in thickness) and dipping beneath the Young Bay Mud to the north and the west. Yerba Buena Mud (also commonly called Old Bay Mud) lies beneath the San Antonio formation.

Due to site elevations and proximity to the San Francisco Bay, the site has relatively shallow groundwater. Based on historic groundwater measurements, we have assumed the groundwater is approximately 4 feet below existing grade in the analyses performed for the site.

Much of the existing fill and some of the Merritt Sand deposits are potentially liquefiable. The Young Bay Mud deposits are highly compressible under loads associated with fill and buildings. The Young Bay Mud is also soft, typically leading to relatively low stability of cuts and slopes as well as low bearing capacity.

GEOTECHNICAL CONSIDERATIONS

Based on the references provided, the main geotechnical concerns for the proposed site development include: (1) stability of the north shoreline, (2) liquefaction, (3) compressible soils and (4) underground utility construction. These concerns are discussed below and should be considered in the initial planning for the project site. A design-level geotechnical analysis should be performed as part of the design process.

North Shoreline Slope Stability

The geotechnical investigation report prepared by Subsurface Consultants Incorporated (SCI) for the Oakland Harbor Navigation Improvement Project at the Port of Oakland (Reference 3) analyzed the proposed deepening and widening of the Inner and Outer Harbor shipping channels and included an evaluation and discussion of that project's impact on adjacent land. The Port's shipping channel deepening project was completed in 2009. A portion of the deepened channel is adjacent to the north shore of the Alameda Point project site.

Reference 3 presents static slope stability analyses performed using limit equilibrium theory to locate the minimum factor of safety and critical slip surface. These analyses were performed using Bishop's Simplified Method and the Spencer Method. Liquefaction analyses were performed using the procedures outlined by Seed, et al. (1984). Lateral spreading was investigated using the Bartlett and Youd method (1995) and seismic slope stability due to inertial forces was analyzed using the method outlined by Makdisi and Seed (1978).

Three levels of seismic design criteria were used in this investigation. Levels 1, 2, and 3 correspond to ground shaking with a 50-, 20-, and 10-percent probability of exceedance in 50 years, and correspond to peak ground accelerations (PGA) of 0.29g, 0.45g, and 0.57g, respectively. A Magnitude 7¼ to 7½ earthquake was assumed for these analyses.

Two cross sections, I-I' and J-J', were analyzed which encroach into a portion of the north shoreline of the proposed Alameda Point project, and the results are presented in Reference 3. The report concluded that the static stability of cross section I-I' was marginal and the seismic performance was poor with very large deformations at all seismic levels. Mitigation in the form of shoreline excavation, ground improvement, rock dikes, and/or bulkheads was recommended. Alternatively, the report suggests moving the channel 25 feet north. The seismic performance of cross section J-J' was concluded to be good at the channel limit but poor at the shoreline. Since the dredging of the channel had a limited effect on the stability of cross section J-J', no mitigation was recommended.

Reference 3 also includes analyses of the northern shoreline stability to the west of the mapped development area. Three additional cross sections, F-F', G-G', and H-H' were evaluated using the methodologies discussed above. The stability was evaluated for both deep failures that would propagate (global failure) on to land as well as localized failures of the cut slope. The previous study indicates that, under static loading, the stability for global failures is relatively high with calculated factors of safety between 1.7 and 2.1, but localized stability of the dredged cut would be slightly above marginal with an approximate factor of safety of 1.3 for all three cross-sections. Under seismic loading, the previous study predicted displacement of the slope (both global and local) for all three cross sections under all three seismic levels. The predicted displacements range from as little as 1 foot to greater than 10 feet of displacement. In all three cross sections, the predicted seismic slope displacements are greater for the localized failure surfaces yet still relatively large for the global failure surfaces.

Based on our understanding of the channel deepening project, no mitigation was performed along the north shore of Alameda Point to improve slope stability.

Limited Slope Stability Analysis

Utilizing information from Reference 3, we analyzed the slope stability of cross sections I-I' and J-J' to verify SCI's results. The locations of these cross sections are shown on Figure 1. We performed the analyses using the computer program SLIDE© (Version 6). SLIDE© is a limit equilibrium program that allows the user various search routines to locate the minimum factor of safety and critical slip surface. We choose the Spencer Method and circular and non-circular searching algorithms for our analysis. We performed seismic deformation analysis on these cross sections, based on the method of Bray and Travasarou (2007) in keeping with the guidelines of the California Geological Survey presented in Special Publication 117A (SP117A). In our analysis, we used the shear strength parameters specified in Reference 3.

Our slope stability calculations indicate that these slopes within the study area are probably marginally stable under current conditions. Any new loads from fill placement or buildings within 50 feet of the northern shoreline would likely have an impact on static slope stability. The calculated seismic slope deformations are in the range (15cm to 100cm) that would be considered potentially seismically "unstable" under SP117A. According to the guidelines, such deformation "may be sufficient to cause serious ground cracking or enough strength loss to result in continuing (post-seismic) failure." Deformations could extend more than 1,000 feet from the shore.

To the west of the study area, the existing slopes appear to be stable under the current conditions but could experience significant deformations (up to 7 feet) under seismic shaking similar to the design earthquake for the site. The distance the deformation could extend is likely smaller than near the development area.

The slope stability results from this study and Reference 3 are included in the Appendix.

Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded fine sands below the groundwater table. Empirical evidence indicates that loose fine-grained soil including low plasticity silt and clay is also potentially liquefiable. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop and liquefaction of susceptible soil to occur. If liquefaction occurs, and if the soil consolidates following liquefaction, then ground settlement and surface deformation may occur. The previous explorations at the site encountered sand and silty sand deposits that could potentially liquefy under seismic loading.

Shallow liquefiable soil is most likely to vent to the surface in the form of sand boils. Sand boils, if they occur, can result in localized voids in the subsurface and bearing failure of shallow foundations and utilities. Sand boils were observed in portions of the Naval Air Station Alameda in the 1989 Loma Prieta Earthquake.

We performed an evaluation of liquefaction potential on selected existing CPT data with the software program Cliq (version 1.7.1.6) applying the methodologies published by NCEER in 1998 and by Moss in 2006. We also analyzed selected existing boring data with the methodologies published by Youd et al. in 2001, Seed et al. in 2003 and Idriss and Boulanger in 2008. We assumed a groundwater level of 4 feet below existing ground surface, a peak ground acceleration (PGA) of 0.4g, and a moment magnitude (M_w) of 7.3. The PGA value corresponds to the 2010 California Building Code seismic design parameters. We evaluated the liquefaction potential for the soil encountered below the assumed water table. The results indicate that sand and silty sand fill material and native deposits are potentially liquefiable down to 40 feet below existing grades. Our analyses also indicate that the potentially liquefiable soil could settle as much 11 inches. Lateral spreading along the northern shoreline is likely following a design level earthquake. A plan showing the depth of liquefiable soil material is provided as Figure 2.

Liquefaction Mitigation

The amount of potential liquefaction settlement and lateral spreading are greater than typical structures and infrastructure can tolerate without mitigation. Ground improvement techniques will likely be necessary to reduce the liquefaction potential of the sandy deposits at the project site to levels that improvements can be designed to tolerate. Liquefiable soil can be mitigated by either dynamic impact/vibration to densify the soil or mixing with cement to create zones of non-liquefiable soil. The success of dynamic impact methods depends on the fines content of the sand and the depth of the liquefiable material.

- Deep Dynamic Compaction

Deep dynamic compaction (DDC) tends to be the most cost-effective method of liquefaction mitigation, where appropriate. DDC imparts impact energy to the soil by dropping a 10- to 15-ton weight from a height of 16 to 50 feet. Since interlayered clay deposits within the liquefiable soil can absorb the dynamic energy and reduce the effectiveness of the ground improvement, DDC is most effective only to depths as much as 35 feet below grade in sandy soil.

Because the method consists of dropping a significant weight from a significant height, DDC results in significant noise and vibration. Since, the vibration impacts typical of DDC will likely cause damage to adjacent structures and improvements, an appropriate setback should be established. DDC should begin in a portion of the site away from existing structures and improvements and vibrations should be monitored to establish a safe setback. Pre- and post-construction surveys of adjacent improvements conditions should be performed to establish

if any damage was caused by DDC. A second ground improvement method may be necessary within any setback area. DDC should not be used over any existing utilities.

- Rapid Impact Compaction

An alternative to DDC is rapid impact compaction (RIC), which is a proprietary densification method where a 7- to 8-ton weight is dropped from 3 to 4 feet high on an approximately 5-foot-diameter hammer head. Because the energy imparted in RIC is significantly less than DDC, it can be used in closer proximity to existing structures and improvements. RIC is most effective in areas were the depth of the liquefiable material is 15 feet or less below the ground surface. Because the treated area is less than with DDC, RIC typically takes longer to treat an area and typically has a higher cost per square foot of area treated.

- Vibratory Replacement

Vibratory replacement methods densify the potentially liquefiable soil by inserting a vibrating probe into the ground and backfilling the shaft created with gravel. This method creates stone columns with densified soil between. The amount of vibration from this method is significantly less than with DDC and the depth of possible treatment is typically at least 35 feet. Unlike DDC and RIC, this method is not performed across the entire project footprint but on a grid of columns with equal spacing across the site. The spacing of the grid would be determined as part of a design-build process.

- Soil/cement Mixing

Soil/cement mixing includes numerous proprietary methods including grouting, grout-mixing, and deep soil mixing. Each of these methods involves mixing the subsurface soil with cement and water to create columns of stiffened soil. The columns can be oriented as individual columns or overlapped to create walls around unimproved soil. The untreated soil is not densified by this technique. This ground improvement method relies on the improved stiffness of the columns to raise the composite stiffness of the site and reduce liquefaction by concentrating the cyclic stresses imparted by the seismic event on the columns and reducing the increase in pore pressure in the soil.

This method of ground improvement results in significantly reduced construction vibrations versus the other alternatives. This method does result in spoils that will be rich in cement; because import is expected at this site, spoils could be mixed with onsite soil to reduce the cement content and used as structural fill once the cement has cured; using spoils as engineered fill will potentially improve performance as a stiffened cap can be constructed to assist in transferring loads to the individual columns. Depending on cement concentration and hydration time, the reaction of cement in the spoils could make conventional soil compaction techniques difficult. If spoils are used as structural fill, we recommend using a method specification to check that appropriate degrees of compaction are achieved.

Compressible Soil

Soft, highly compressible Yong Bay Mud deposits were encountered in the previous explorations at the project site. A plan showing the depth of the base of the Young Bay Mud is provided as Figure 3. The locations and thicknesses of these deposits are variable, ranging from nil to over 130 feet in thickness. The Yong Bay Mud can settle due to loading from any new fill or from new structures constructed at the site. The amount of settlement is a factor of load and thickness of Young Bay Mud. Assuming the Young Bay Mud is normally consolidated, settlement can be as great a ½ foot for each foot of fill placed over the thickest areas of Young Bay Mud. While the majority of settlement from new loads will happen in the first 1 to 2 years after construction, in the areas of the thickest Young Bay Mud, settlement can continue for a period of 50 years or more.

Compressible Soil Mitigation

Depending on the type of buildings planned at the project site, mitigation of the compressible Young Bay Bud deposits may be feasible. One measure that can be used to mitigate the loading from small, relatively lightweight structures is pre-consolidation of compressible material through a surcharge program. Surcharge fill is placed above design grade elevations in areas of the site where pre-consolidation measures are necessary to reduce settlement. The surcharge fill remains in place for a period sufficient to allow the desired degree of consolidation to be achieved, such that the risk of settlement is sufficiently reduced for the planned structure. Surcharging will induce some settlement in adjacent areas; therefore, it may not be feasible to use surcharge as a compressible soil mitigation method in areas near existing structures and utilities. Likewise, surcharging of initial phases of construction should be placed wider than the footprint of the construction area so that subsequent phases of surcharge do not cause settlement of already constructed areas. For planning purposes, we recommend assuming that surcharge areas of initial phases should be overbuilt by at least 20 feet laterally from the improvement area.

The amount of time necessary to effectively mitigate compressible soil through surcharge is directly related to the thickness of the compressible soil deposit. Where the Young Bay Mud is thicker than about 20 feet, it is likely that wick drains may be desired to shorten the drainage path of the compressible deposits and accelerate the surcharge program.

A surcharge program is generally not efficient for structures with bearing pressures over 750 to 1,000 pounds per square foot. In these cases deep foundation systems deriving support from below the Young Bay Mud could be suitable at the project site. Where deep foundations are used, utilities should incorporate flexible connections as the building will not settle with the surrounding soil.

Underground Utilities

Utility Trench Shoring

Due to the soft nature of the Young Bay Mud, excavations that extend into Young Bay Mud deposits may become unstable. Installation of temporary sheetpiles or the use of a shield or continuous hydraulic skeleton shoring should be anticipated for excavations that extend below a depth of about 3 to 5 feet.

Trench Dewatering

Shallow groundwater is expected at the site and trench excavations may encounter perched groundwater. Therefore, utility trench excavations may require temporary dewatering during construction to keep the excavation and working areas reasonably dry. In general, excavations should be dewatered such that water levels are maintained at least 2 feet below the bottom of the excavation prior to and continuously during shoring installation and the backfill process to control the tendency for the bottom of the excavation to heave under hydrostatic pressures and to reduce inflow of soil or water from beneath temporary shoring. We anticipate that dewatering for underground utility construction will be accomplished by pumping from sumps.

Utility trenches adjacent to existing improvements should include a low permeability cutoff to reduce the risk of inadvertent groundwater flow along permeable bedding or backfill. In these areas dewatering may not be an option; therefore, a relatively impervious shoring system of tight interlocking sheet piles, or other impervious wall type, can be utilized to reduce infiltration during construction.

In addition, possibility of encountering contaminated soil and groundwater should be considered during underground construction.

LAND PLANNING ZONES

The limits of the land planning zones discussed below are presented on Figure 4.

North Shore Line

We understand that a significant setback from the north shore is not feasible; therefore, strengthening of the shoreline will be needed to reduce potential lateral displacement. The most cost effective shoreline stabilization measure would likely be performing ground improvement such as soil/cement mixing. Because both the liquefiable fill and Young Bay Mud impact the seismic slope stability, the soil/cement mixing will need to extend about 40 feet below the ground surface to the bottom of the Young Bay Mud layer. Based on similar projects, we estimate that to appropriately improve shoreline stability the soil treatment may need to be performed on 15 to 30 percent of the soil volume over an area between 20 to 30 feet wide. Other shoreline improvement measures, such as a levee and flood protection system could be

constructed in conjunction with the improvement area. An alternative to soil/cement mixing would be construction of a structure, such as a bulkhead wall.

We understand that a levee has been proposed as part of the flood protection system on the northern shoreline. The levee embankment should have a crest 12 feet wide with side slopes of approximately 3:1 (horizontal:vertical). We recommend that the material used for embankment construction consist of soil with at least 15 percent passing the No. 200 sieve and no particles greater than 6 inches in maximum dimension.

Adaptive Reuse Area

We understand that some portions of the site are planned for adaptive reuse. In these areas, liquefaction mitigation measures will be constrained by existing structures and utilities. Ground improvement techniques will not be available for existing buildings; therefore, potential liquefaction induced settlement must be mitigated structurally. Where new utilities are to be installed, RIC could be used to densify the top 15 feet of liquefiable material, and the utilities could be designed to withstand settlement up to 8 inches and differential settlement up to 4 inches. Alternatively, vibratory replacement or soil/cement mixing could be used in these areas to reduce settlement of utilities and other improvements; total and differential settlement using these approaches would be less than using RIC. Based on typical construction costs, ground improvement using RIC will likely be the most cost efficient solution though other ground improvement methods would be more effective in decreasing potential settlement where liquefiable soil is deeper than 15 feet. Existing utilities that will remain in place can be supported by grouting underneath the utility.

Liquefaction Hazard Area

This area is not planned for adaptive reuse, so DDC will be the most applicable and cost effective liquefaction mitigation method. DDC results in relatively large noise and vibration impacts, so a buffer zone of up to 100 feet may be necessary from any existing structures to minimize impacts. Inside this buffer zone, other ground improvement methods may be necessary.

Liquefaction and Compressible Soil Hazard Area

DDC will also be the most applicable and cost effective liquefaction mitigation method in this area. DDC results in relatively large noise and vibration impacts, so a buffer zone of up to 100 feet may be necessary from any existing structures to minimize impacts. Inside this buffer zone, other ground improvement methods may be necessary.

Structures constructed in this area that have bearing pressures greater than 750 to 1,000 pounds per square foot will likely need to be supported on deep foundations. A surcharge program could be used to mitigate the consolidation settlement caused by the construction of light buildings.

Outside of the building areas, additional fill from grading to raise the site out of the flood plain will also induce consolidation settlement of the Young Bay Mud, and we anticipate that other measures may be necessary to mitigate potential settlement that could adversely affect site improvements (i.e., streets, parking areas, drainage, underground utilities, concrete flatwork, etc.). The selected mitigation will partly depend on what level of risk is acceptable, and could range from: (1) acceptance of settlement risk and periodic maintenance, (2) implementation of a surcharge program to pre-consolidate the soil and reduce long term settlements, (3) use of lightweight fill as compensation load to reduce settlement or (4) critical utilities could be supported on cement/soil mixed columns.

The comments provided in this letter are professional opinions developed in accordance with current standards of geotechnical engineering practice; no warranty is expressed or implied. If you have any questions regarding our letter, please do not hesitate to contact us.

Sincerely,

ENGEO Incorporated

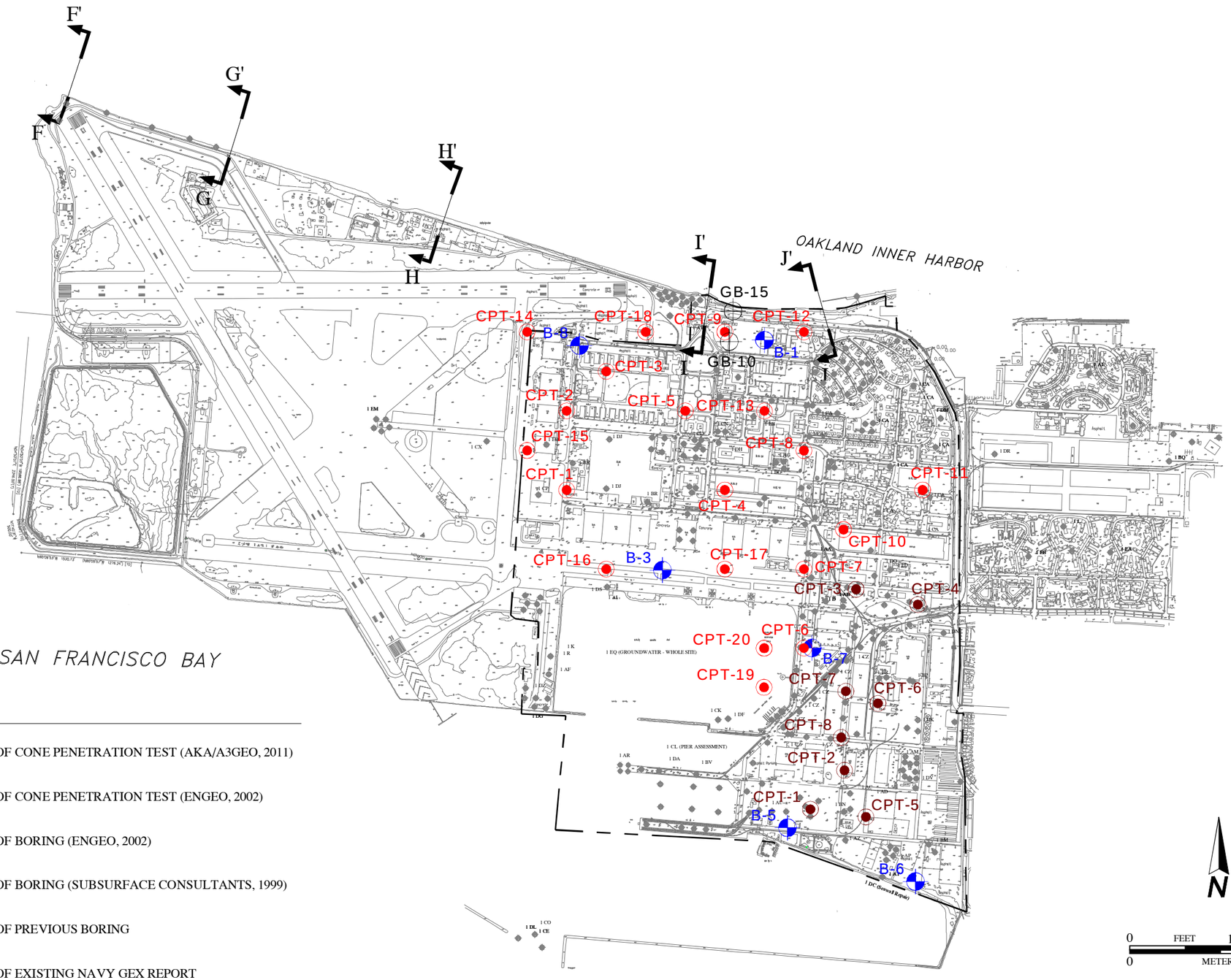
Siobhan O'Reilly-Shah

Jeff Fippin, GE

REGISTERED PROFESSIONAL ENGINEER
JEFFREY A. FIPPIN
No. 2631
Exp. 6/30/2013
GEOTECHNICAL
STATE OF CALIFORNIA

Daniel S. Haynosch, GE
sors/jf/dsh/jf

Attachments: Figure 1 - Site Plan
Figure 2 - Depth of Potentially Liquefiable Soil
Figure 3 - Thickness of Young Bay Mud
Figure 4 – Preliminary Constraints Mapping Based on Land Planning Zones
Appendix – Limited Slope Stability Calculations



EXPLANATION

CPT-8 — APPROXIMATE LOCATION OF CONE PENETRATION TEST (AKA/A3GEO, 2011)

CPT-20 — APPROXIMATE LOCATION OF CONE PENETRATION TEST (ENGEO, 2002)

B-8 — APPROXIMATE LOCATION OF BORING (ENGEO, 2002)

GB-15 — APPROXIMATE LOCATION OF BORING (SUBSURFACE CONSULTANTS, 1999)

APPROXIMATE LOCATION OF PREVIOUS BORING

1 AE — APPROXIMATE LOCATION OF EXISTING NAVY GEX REPORT

J J' — APPROXIMATE LOCATION OF CROSS SECTION EVALUATED BY SUBSURFACE CONSULTANTS

BASE MAP SOURCE: CARLSON, BARBEE & GIBSON

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SITE PLAN

ALAMEDA POINT - INFRASTRUCTURE PLANNING

ALAMEDA, CALIFORNIA

PROJECT NO.: 5687.100.104	FIGURE NO
SCALE: AS SHOWN	1
DRAWN BY: SRP	CHECKED BY: JF

ORIGINAL FIGURE PRINTED IN COLOR

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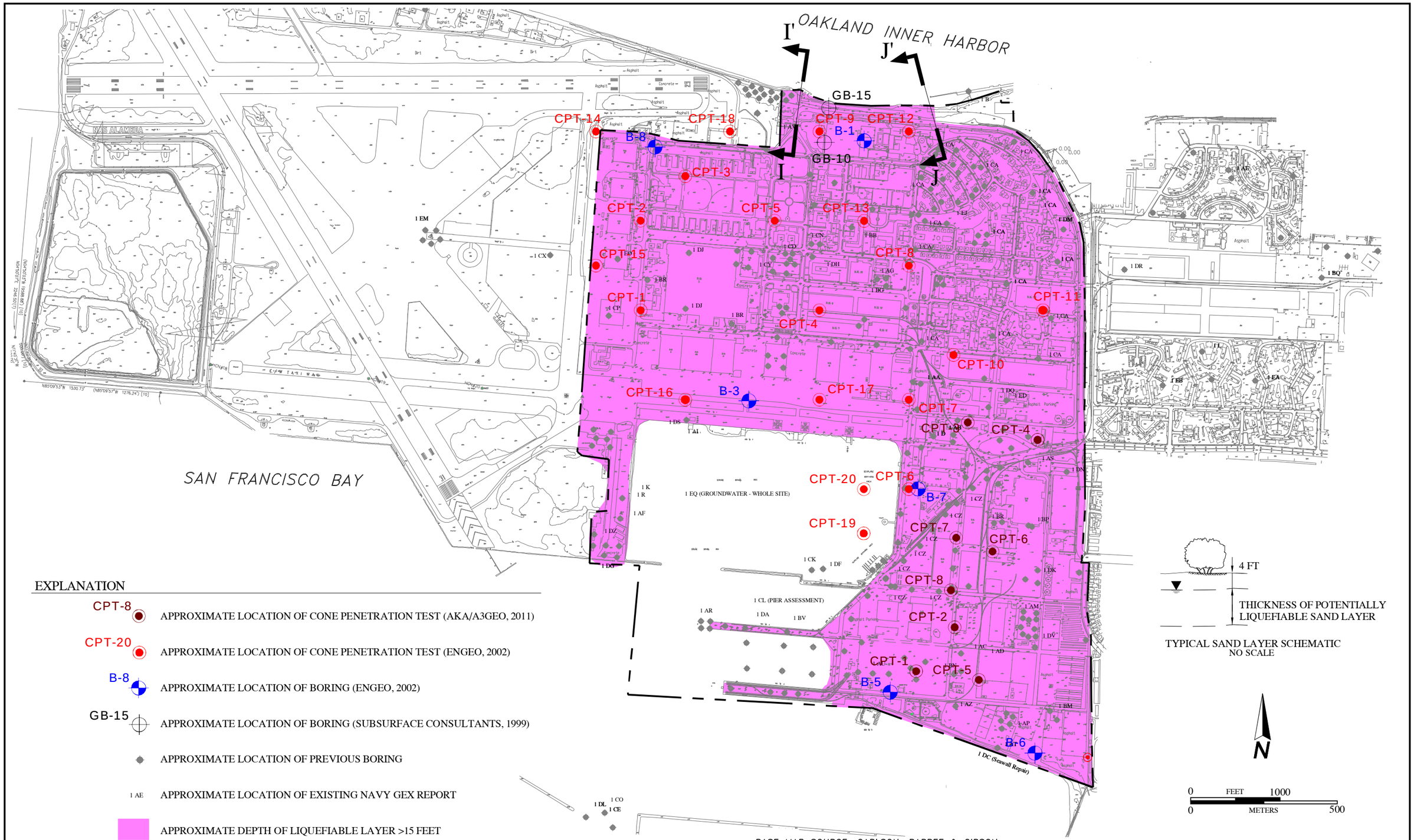

EXPLANATION

- CPT-8 — APPROXIMATE LOCATION OF CONE PENETRATION TEST (AKA/A3GEO, 2011)
- CPT-20 — APPROXIMATE LOCATION OF CONE PENETRATION TEST (ENGEO, 2002)
- B-8 — APPROXIMATE LOCATION OF BORING (ENGEO, 2002)
- GB-15 — APPROXIMATE LOCATION OF BORING (SUBSURFACE CONSULTANTS, 1999)
- APPROXIMATE LOCATION OF PREVIOUS BORING
- 1 AE — APPROXIMATE LOCATION OF EXISTING NAVY GEX REPORT
- APPROXIMATE DEPTH OF LIQUEFIABLE LAYER >15 FEET
- J — J' APPROXIMATE LOCATION OF CROSS SECTION

BASE MAP SOURCE: CARLSON, BARBEE & GIBSON

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DEPTH OF POTENTIALLY LIQUEFIABLE SOIL
ALAMEDA POINT - INFRASTRUCTURE PLANNING
ALAMEDA, CALIFORNIA

PROJECT NO.: 5687.100.104	FIGURE NO
SCALE: AS SHOWN	2
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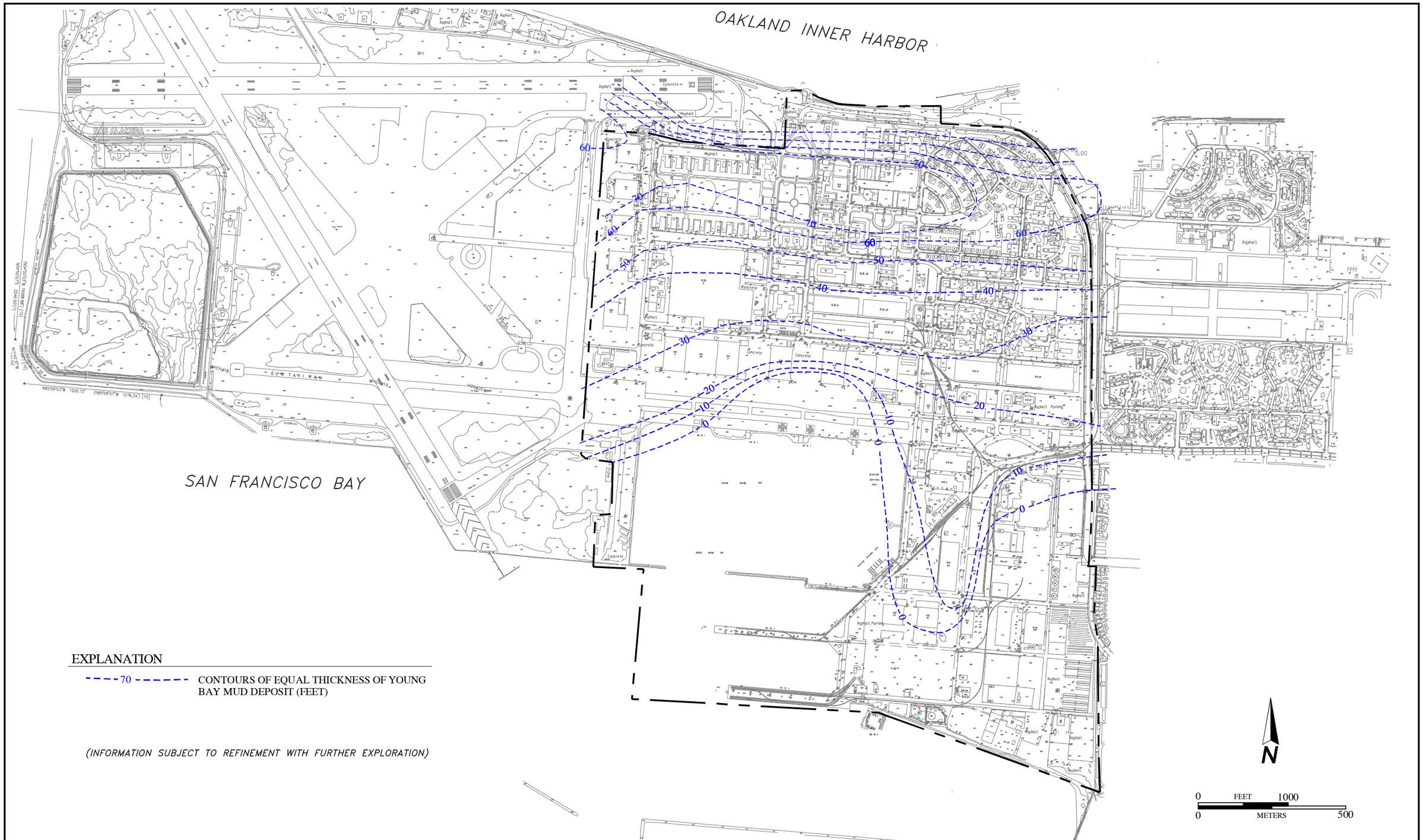

BASE MAP SOURCE: CARLSON, BARBEE & GIBSON

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THICKNESS OF YOUNG BAY MUD

ALAMEDA POINT - INFRASTRUCTURE PLANNING

ALAMEDA, CALIFORNIA

PROJECT NO.: 5687.100.104	FIGURE NO
SCALE: AS SHOWN	3
DRAWN BY: SRP / CHECKED BY: JF	

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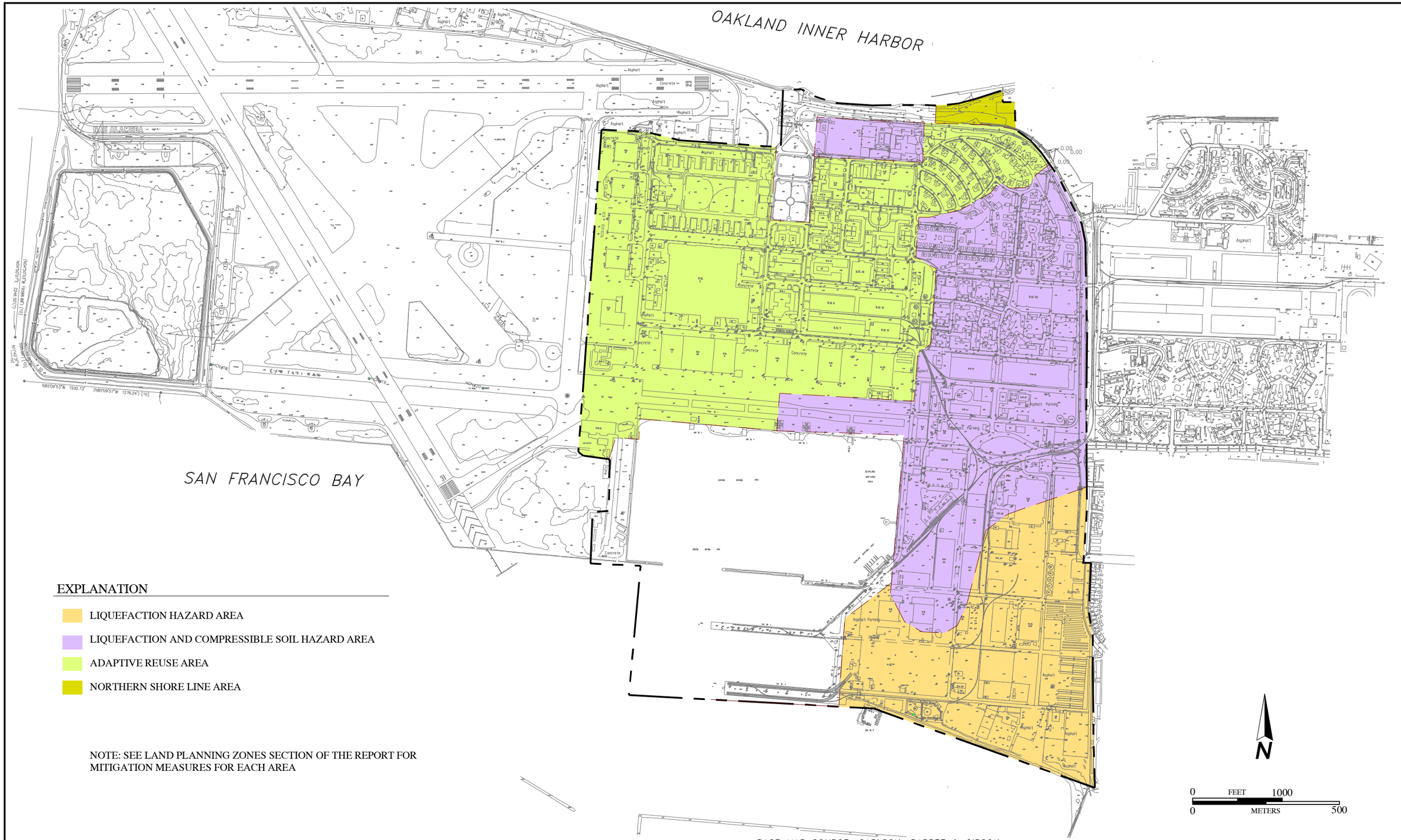

BASE MAP SOURCE: CARLSON, BARBEE & GIBSON

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PRELIMINARY CONSTRAINTS MAPPING
BASED ON LAND PLANNING ZONES
ALAMEDA POINT - INFRASTRUCTURE PLANNING
ALAMEDA, CALIFORNIA

PROJECT NO.: 5687.100.104
SCALE: AS SHOWN
DRAWN BY: SRP
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FIGURE NO
4

ORIGINAL FIGURE PRINTED IN COLOR

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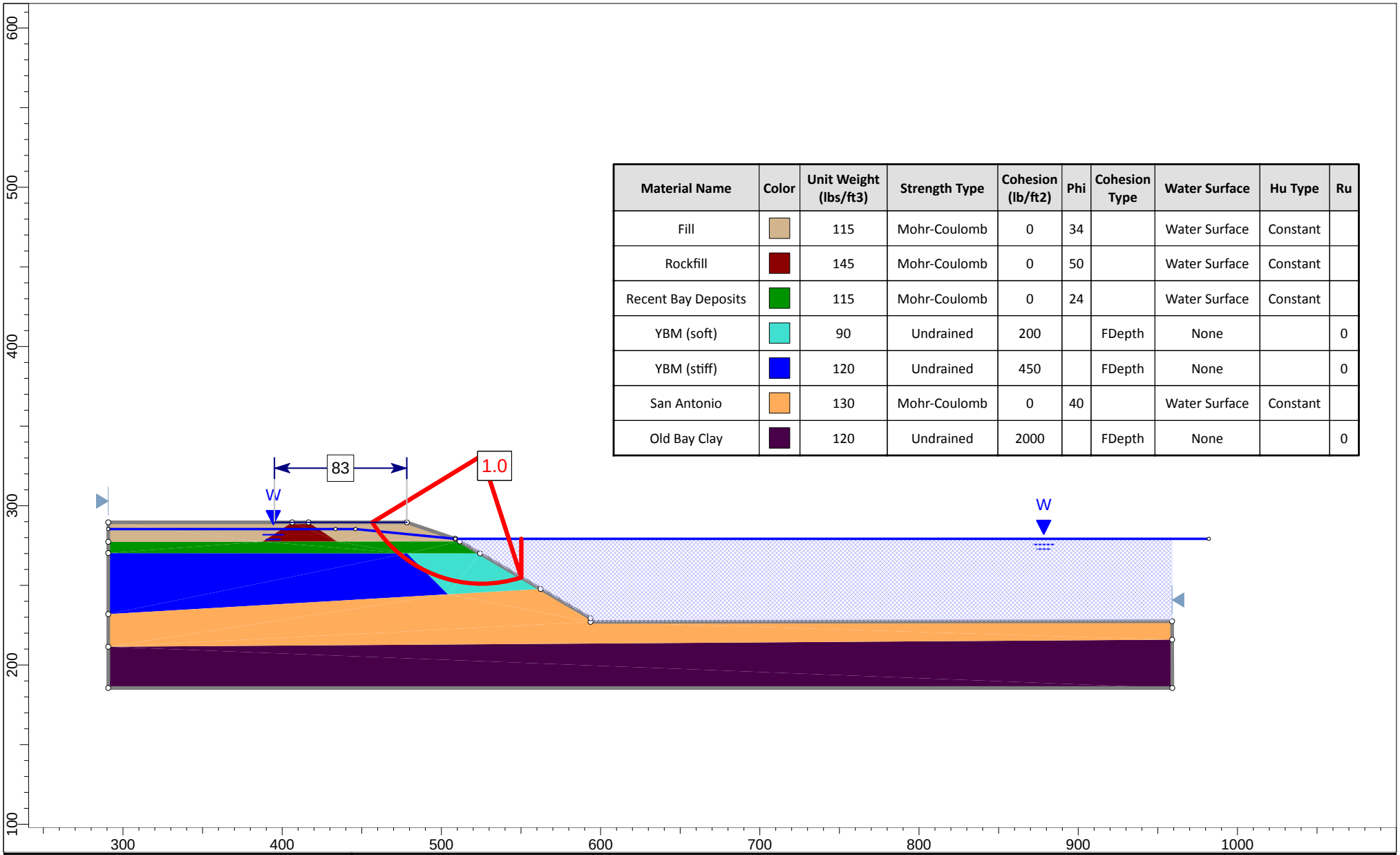
APPENDIX

Limited Slope Stability Calculations

5687.100.104
January 16, 2013
Revised January 30, 2013

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type	Water Surface	Hu Type	Ru
Fill		115	Mohr-Coulomb	0	34		Water Surface	Constant	
Rockfill		145	Mohr-Coulomb	0	50		Water Surface	Constant	
Recent Bay Deposits		115	Mohr-Coulomb	0	24		Water Surface	Constant	
YBM (soft)		90	Undrained	200		FDepth	None		0
YBM (stiff)		120	Undrained	450		FDepth	None		0
San Antonio		130	Mohr-Coulomb	0	40		Water Surface	Constant	
Old Bay Clay		120	Undrained	2000		FDepth	None		0

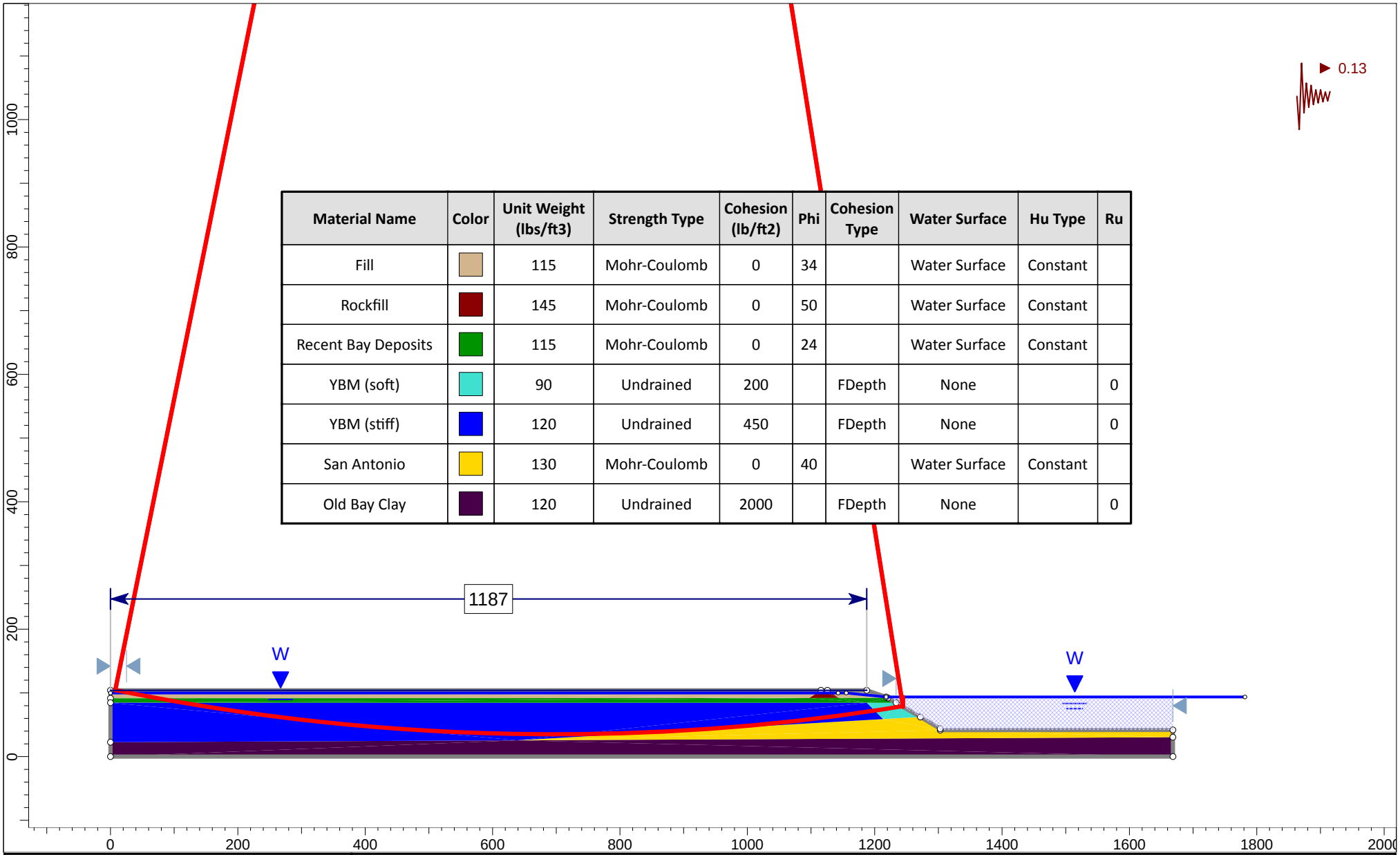


ENGEO Expect Excellence 40 years

SLIDEINTERPRET 6.014

Project	Alameda Point		
Analysis Description	Spencer		
Drawn By	Siobhan O'Reilly-Shah	Scale	1:1000
Company	ENGEO		
Date	12/12/2012, 10:36:05 AM	File Name	Static Slope Stability - xsec I-I'.slim

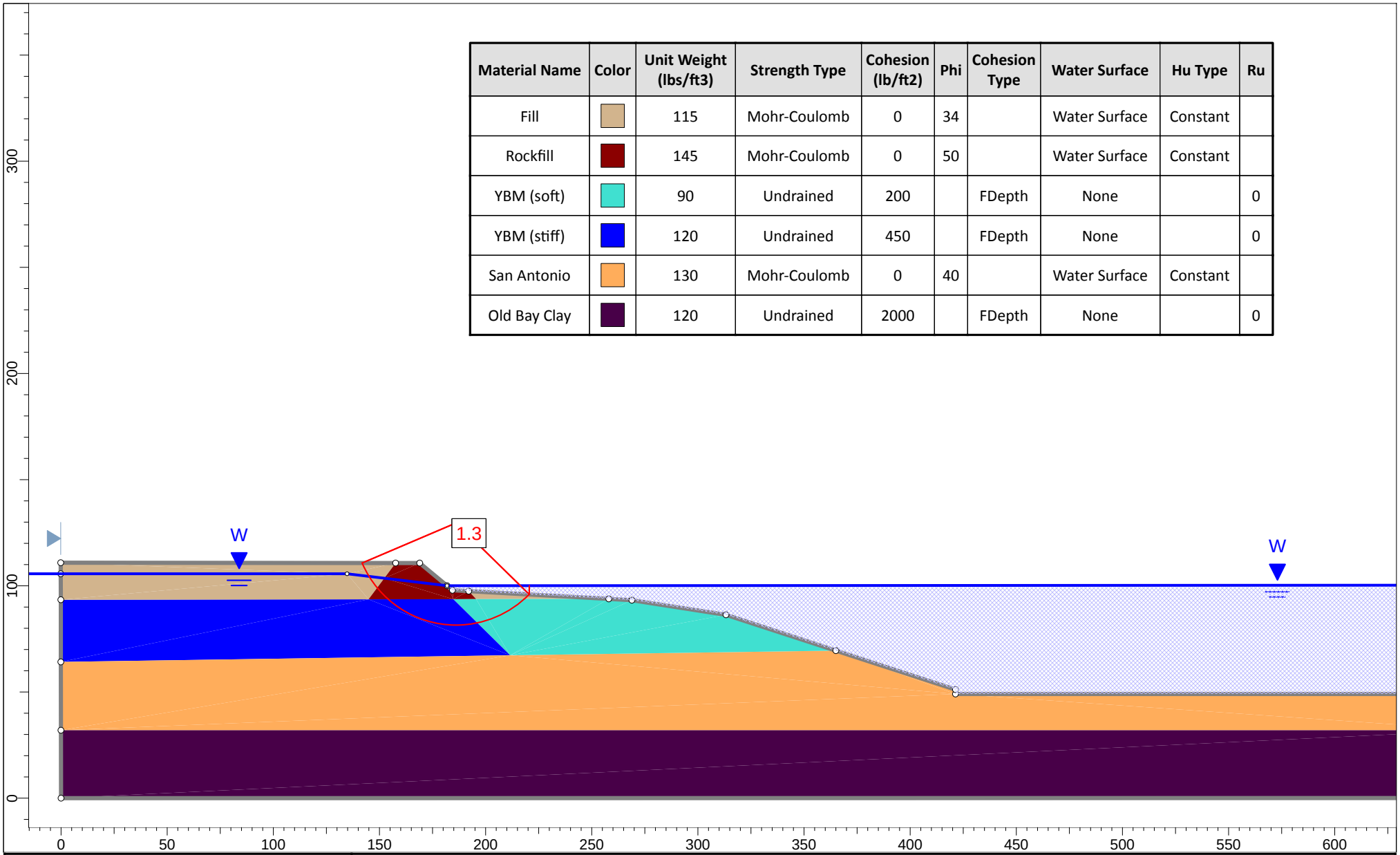
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type	Water Surface	Hu Type	Ru
Fill		115	Mohr-Coulomb	0	34		Water Surface	Constant	
Rockfill		145	Mohr-Coulomb	0	50		Water Surface	Constant	
Recent Bay Deposits		115	Mohr-Coulomb	0	24		Water Surface	Constant	
YBM (soft)		90	Undrained	200		FDepth	None		0
YBM (stiff)		120	Undrained	450		FDepth	None		0
San Antonio		130	Mohr-Coulomb	0	40		Water Surface	Constant	
Old Bay Clay		120	Undrained	2000		FDepth	None		0



ENGEO Expect Excellence 40 years	Project	Alameda Point	
	Analysis Description	Spencer	
	Drawn By: Siobhan O'Reilly-Shah	Scale: 1:2500	Company: ENGEO
	Date: 12/12/2012, 10:36:05 AM		File Name: Pseudo-Static Slope Stability - xsec I-I'.slim

SLIDEINTERPRET 6.014

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type	Water Surface	Hu Type	Ru
Fill		115	Mohr-Coulomb	0	34		Water Surface	Constant	
Rockfill		145	Mohr-Coulomb	0	50		Water Surface	Constant	
YBM (soft)		90	Undrained	200		FDepth	None		0
YBM (stiff)		120	Undrained	450		FDepth	None		0
San Antonio		130	Mohr-Coulomb	0	40		Water Surface	Constant	
Old Bay Clay		120	Undrained	2000		FDepth	None		0

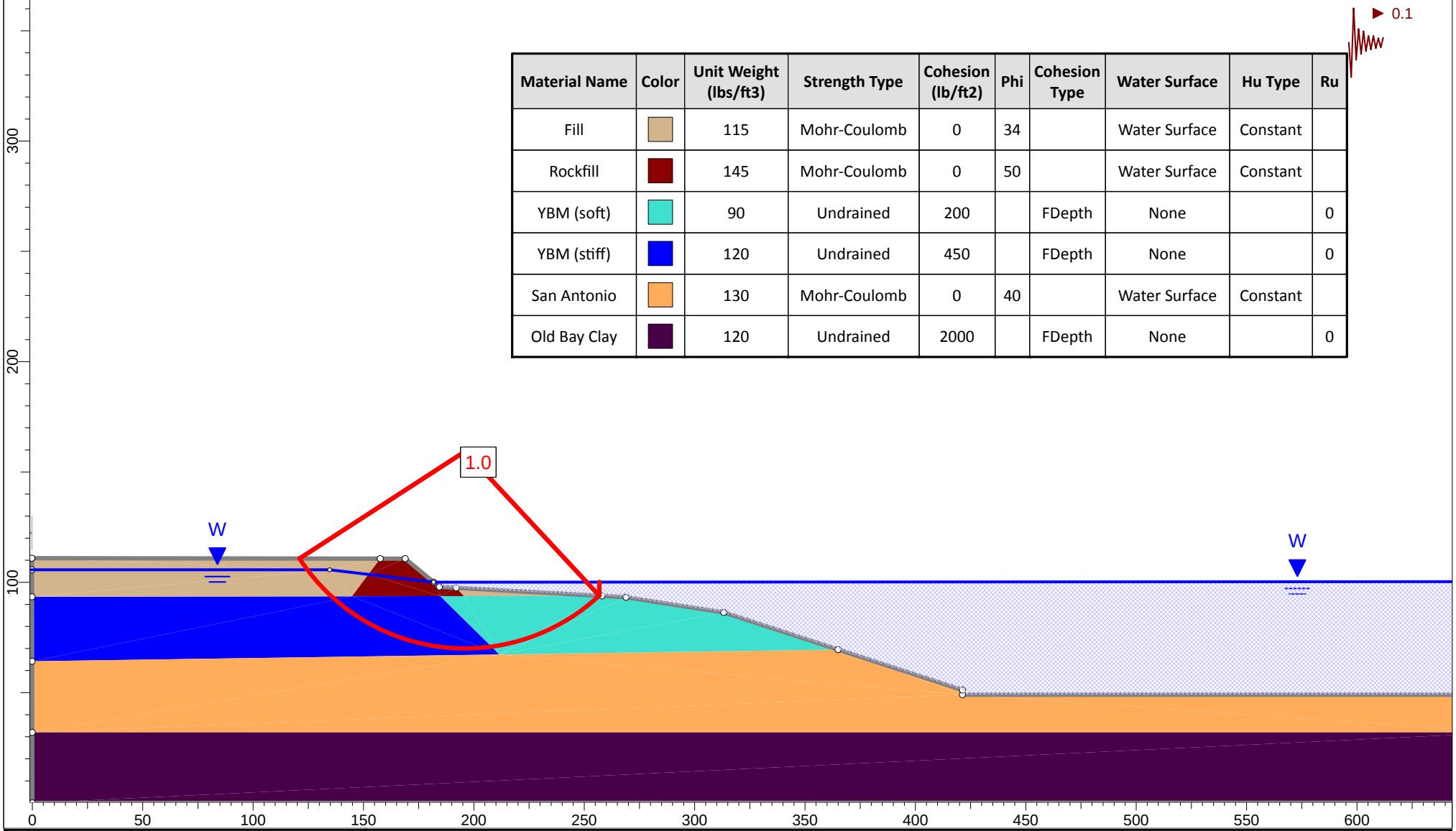


ENGEO Expect Excellence 40 years

SLIDEINTERPRET 6.014

Project	Alameda Point		
Analysis Description	Spencer		
Drawn By	Siobhan O'Reilly-Shah	Scale	1:750
Company	ENGEO		
Date	12/12/2012, 10:36:05 AM	File Name	Static Slope Stability - xsec J-J'.slim

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Cohesion Type	Water Surface	Hu Type	Ru
Fill		115	Mohr-Coulomb	0	34		Water Surface	Constant	
Rockfill		145	Mohr-Coulomb	0	50		Water Surface	Constant	
YBM (soft)		90	Undrained	200		FDepth	None		0
YBM (stiff)		120	Undrained	450		FDepth	None		0
San Antonio		130	Mohr-Coulomb	0	40		Water Surface	Constant	
Old Bay Clay		120	Undrained	2000		FDepth	None		0

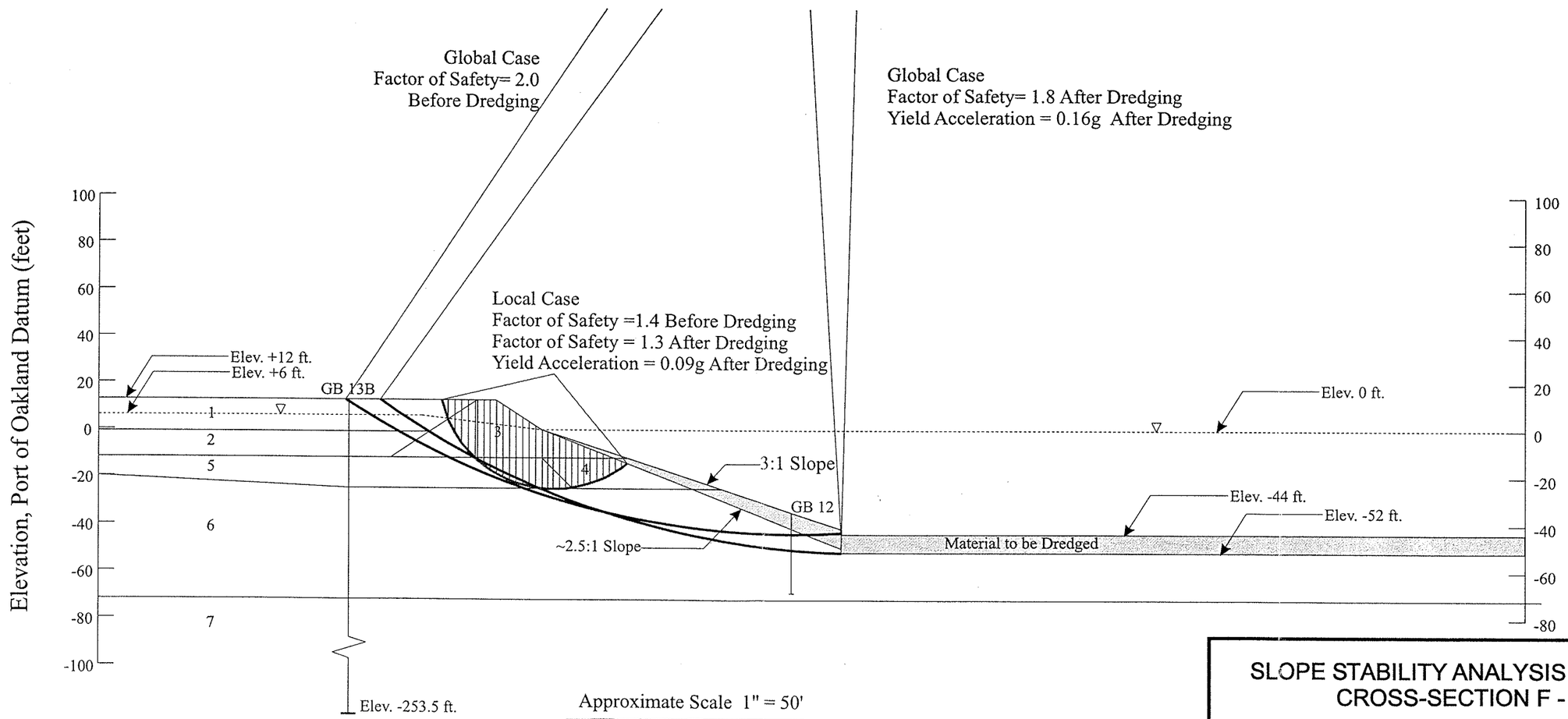


Project	Alameda Point		
Analysis Description	Spencer		
Drawn By: Siobhan O'Reilly-Shah	Scale: 1:750	Company: ENGEO	
Date: 12/12/2012, 10:36:05 AM		File Name: Pseudo-Static Slope Stability - xsecJ-J".slim	

ENGEO Expect Excellence 40 years

SLIDEINTERPRET 6.014

Layer No.	Soil Classification (Lithologic Unit)	Unit Weight (pcf)	Effective Friction Angle (degrees)	Effective Cohesion Intercept (psf)	Undrained Shear Strength (psf)
1	Loose Sand (Fill)	115	30	0	-
2	Soft to Medium Stiff Clay (Fill)	100	-	-	400 at top of layer, increasing 9 psf/ft.
3	Rockfill (Old Training Wall)	145	50	0	-
4	Soft Clay (Young Bay Mud)	95	-	-	250 at El. -12 ft., increasing 11 psf/ft
5	Medium Stiff Clay (Young Bay Mud)	100	-	-	500 at top of layer, increasing 11 psf/ft.
6	Very Dense Sand (San Antonio Formation)	130	40	0	-
7	Very Stiff Clay (Old Bay Mud)	120	-	-	2000 at top of layer, increasing 30 psf/ft



Notes:
1. Yield Acceleration was determined using a post-liquefaction residual strength of 300 psf for the submerged part of layer #1
2. If the new channel limit is moved 25 feet north, the -42 foot channel slope can be extended at a 3:1 slope so that the local stability case is not impacted by the proposed dredging.

Approximate Scale 1" = 50'

0 50 100

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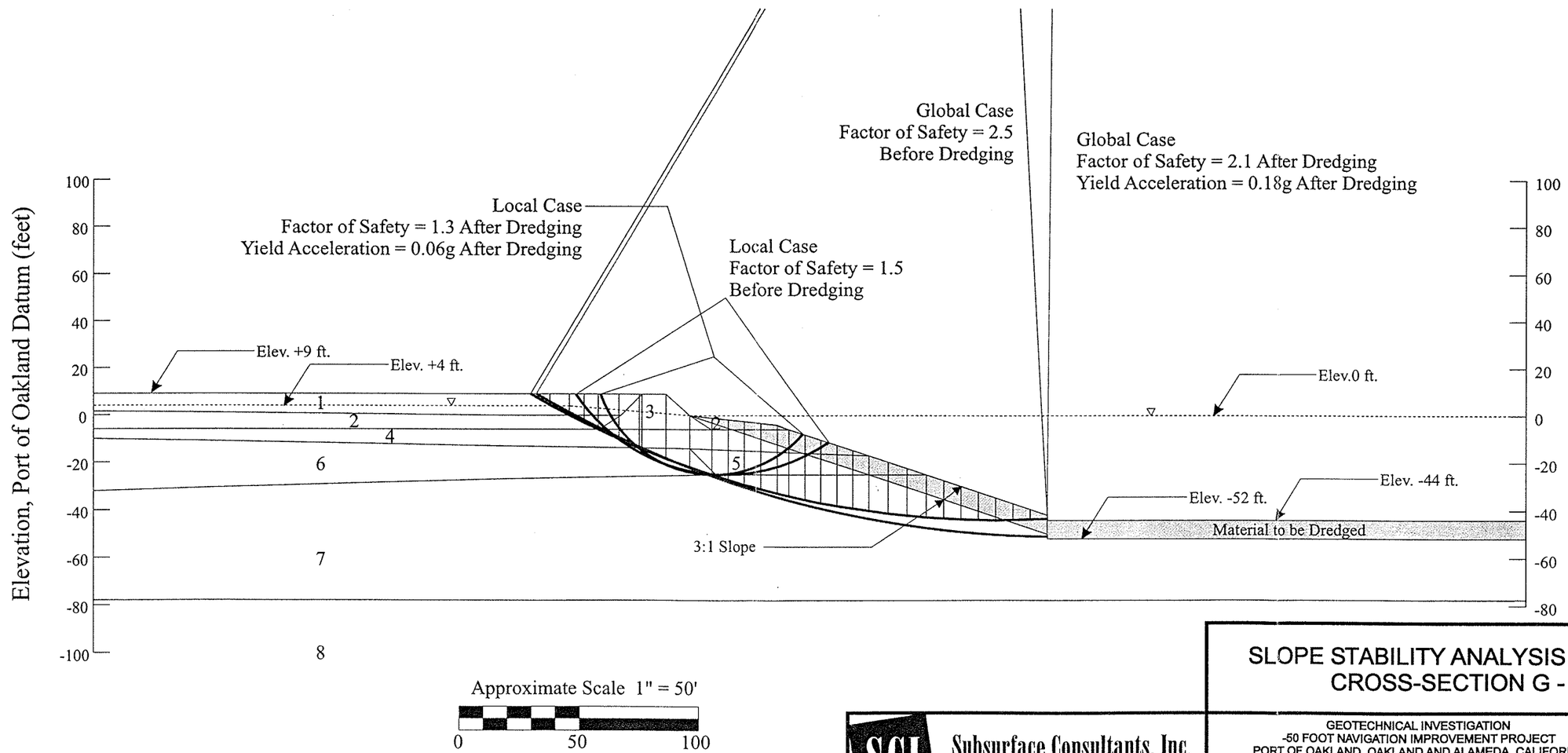
SLOPE STABILITY ANALYSIS RESULTS
CROSS-SECTION F - F'

GEOTECHNICAL INVESTIGATION
-50 FOOT NAVIGATION IMPROVEMENT PROJECT
PORT OF OAKLAND, OAKLAND AND ALAMEDA, CALIFORNIA

JOB NUMBER	DATE	APPROVED
133.007	12/97	

PLATE 37

Layer No.	Soil Classification (Lithologic Unit)	Unit Weight (pcf)	Effective Friction Angle (degrees)	Effective Cohesion Intercept (psf)	Undrained Shear Strength (psf)
1	Loose Sand (Fill)	115	30	0	-
2	Loose Sand (Fill)	115	30	0	-
3	Rockfill (Old Training Wall)	145	50	0	-
4	Interbedded Loose Sand and Soft Clay (Recent Bay Deposits)	115	30	0	-
5	Soft Clay (Young Bay Mud)	95	-	-	250 at El. -14 ft., increasing 11 psf/ft
6	Medium Stiff Clay (Young Bay Mud)	100	-	-	500 at top of layer, increasing 11 psf/ft.
7	Very Dense Sand (San Antonio Formation)	130	40	0	-
8	Very Stiff Clay (Old Bay Mud)	120	-	-	2000 at top of layer, increasing 30 psf/ft



Approximate Scale 1" = 50'

0 50 100

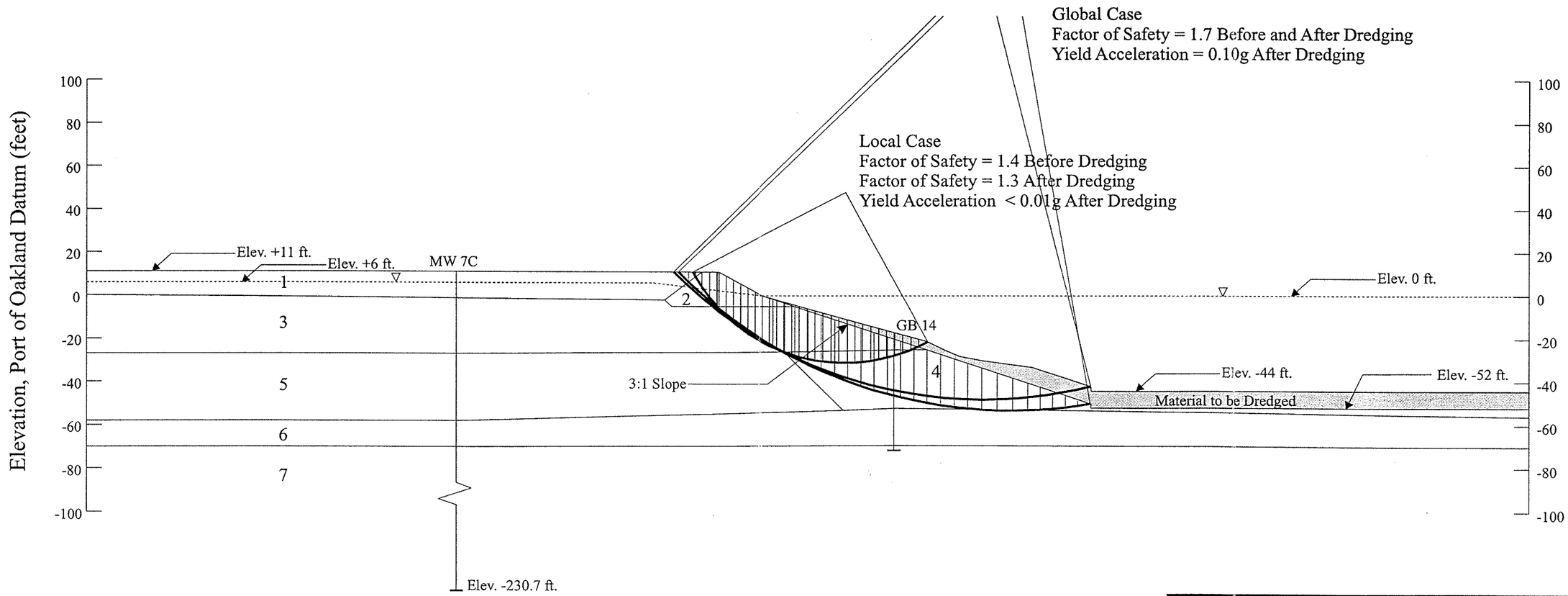
SLOPE STABILITY ANALYSIS RESULTS
CROSS-SECTION G - G'

Subsurface Consultants, Inc.
Geotechnical & Environmental Engineers

GEOTECHNICAL INVESTIGATION
-50 FOOT NAVIGATION IMPROVEMENT PROJECT
PORT OF OAKLAND, OAKLAND AND ALAMEDA, CALIFORNIA

JOB NUMBER	DATE	APPROVED	PLATE
133.007	12/97		38

Layer No.	Soil Classification (Lithologic Unit)	Unit Weight (pcf)	Effective Friction Angle (degrees)	Effective Cohesion Intercept (degrees)	Undrained Shear Strength (psf)
1	Loose Sand (Fill)	115	30	0	-
2	Rockfill (Old Training Wall)	145	50	0	-
3	Interbedded Loose Sand and Soft Clay (Recent Bay Deposits)	115	30	0	-
4	Soft Clay (Young Bay Mud)	100	-	-	300 at El. -27 ft., increasing 11 psf/ft.
5	Interbedded Medium Stiff Clay and Medium Dense Clayey Sand (Young Bay Mud and San Antonio Formations)	105	-	-	500 at El. -27 ft., increasing 11 psf/ft.
6	Very Dense Sand (San Antonio Formation)	130	40	0	
7	Very Stiff Clay (Old Bay Clay)	120	-	-	2000 at top of layer, increasing 30 psf/ft.



Approximate Scale 1" = 50'

0 50 100

SLOPE STABILITY ANALYSIS RESULTS
CROSS-SECTION H - H'

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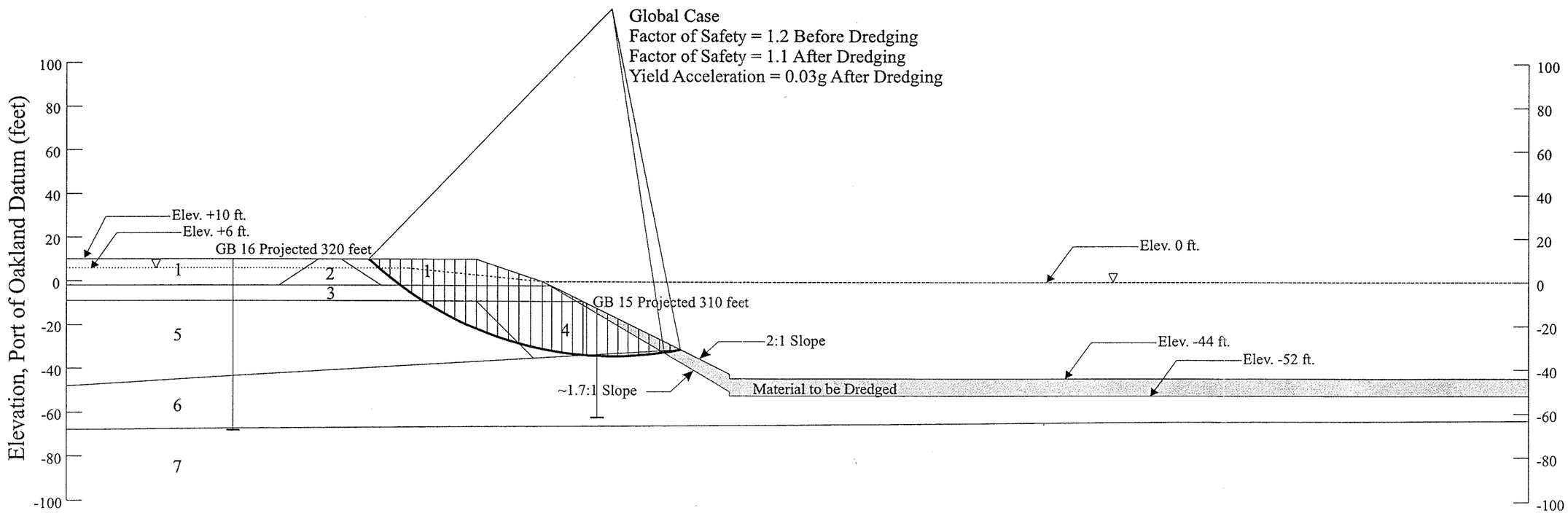
GEOTECHNICAL INVESTIGATION
-50 FOOT NAVIGATION IMPROVEMENT PROJECT
PORT OF OAKLAND, OAKLAND AND ALAMEDA, CALIFORNIA

JOB NUMBER	DATE	APPROVED
133.007	12/97	

PLATE 39

Layer No.	Soil Classification (Lithologic Unit)	Unit Weight (pcf)	Effective Friction Angle (degrees)	Effective Cohesion Intercept (degrees)	Undrained Shear Strength (psf)
1	Medium Dense to Dense Sand (Fill)	115	34	0	-
2	Rockfill (Old Training Wall)	145	50	0	-
3	Medium Dense to Dense Sand (Recent Bay Deposits)	115	34	0	-
4	Soft Clay (Young Bay Mud)	90	-		200 at El. -27 ft., increasing 11 psf/ft.
5	Medium Stiff Clay (Young Bay Mud)	95	-	-	450 at El. -27 ft., increasing 11 psf/ft.
6	Very Dense Sand (San Antonio Formation)	130	40	0	-
7	Very Stiff Clay (Old Bay Clay)	120	-	-	2000 at top of layer, increasing 30 psf/ft.



Approximate Scale 1" = 50'

0 50 100

SLOPE STABILITY ANALYSIS RESULTS
CROSS-SECTION I - I'

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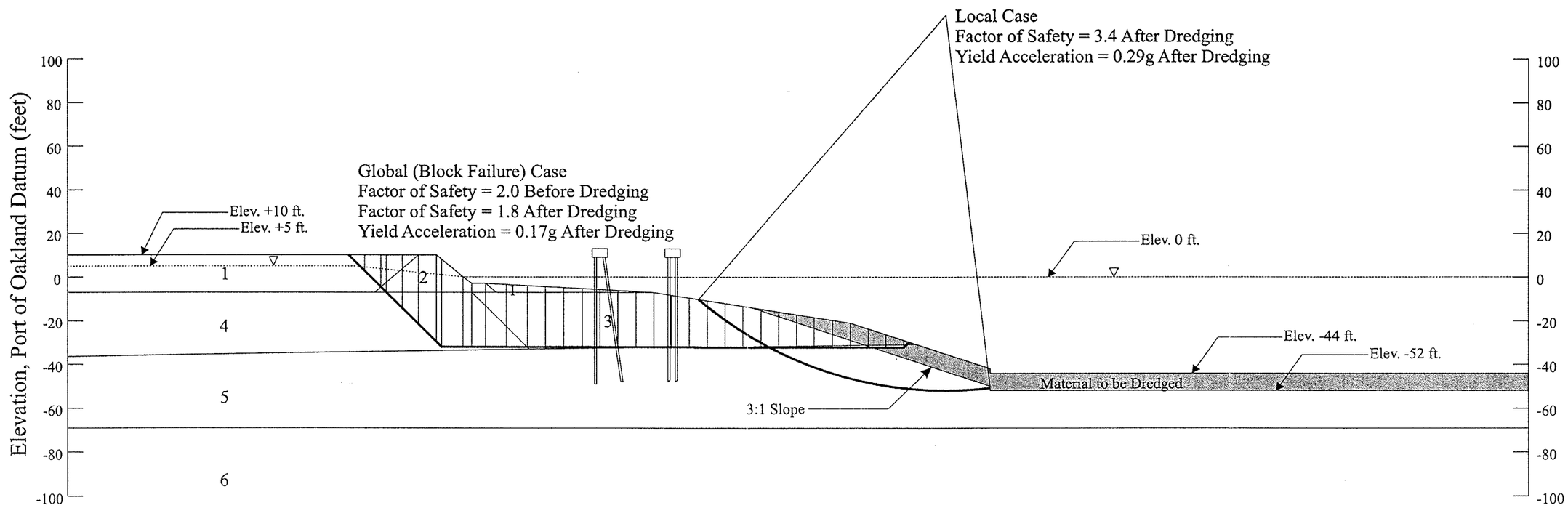
GEOTECHNICAL INVESTIGATION
-50 FOOT NAVIGATION IMPROVEMENT PROJECT
PORT OF OAKLAND, OAKLAND AND ALAMEDA, CALIFORNIA

JOB NUMBER	DATE	APPROVED	PLATE
133.007	12/97		40

Layer No.	Soil Classification (Lithologic Unit)	Unit Weight (pcf)	Effective Friction Angle (degrees)	Effective Cohesion Intercept (degrees)	Undrained Shear Strength (psf)
1	Loose to Medium Dense Sand (Fill)	115	30	0	-
2	Rockfill (Old Training Wall)	145	50	0	-
3	Soft Clay (Young Bay Mud)	90	-		170 at El. -7 ft., increasing 11 psf/ft.
4	Medium Stiff Clay (Young Bay Mud)	95	-	-	350 at El. -7 ft., increasing 11 psf/ft.
5	Very Dense Sand (San Antonio Formation)	130	40	0	-
6	Very Stiff Clay (Old Bay Clay)	120	-	-	2000 at top of layer, increasing 30 psf/ft.



Approximate Scale 1" = 50'

0 50 100

SLOPE STABILITY ANALYSIS RESULTS
CROSS-SECTION J - J'

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GEOTECHNICAL INVESTIGATION
-50 FOOT NAVIGATION IMPROVEMENT PROJECT
PORT OF OAKLAND, OAKLAND AND ALAMEDA, CALIFORNIA

JOB NUMBER	DATE	APPROVED
133.007	12/97	

PLATE 41