Appendix G: Geotechnical

For

WESTMINSTER, EAST GARDEN GROVE

FLOOD RISK MANAGEMENT STUDY

September 2019

US Army Corps of Engineers
Chicago District

1 Scope of Work

The purpose of this appendix is to provide results from the Geotechnical Design effort. Design data and calculations were developed sufficiently to determine the technical and economic feasibility of each alternative and in the event that project is authorized, to provide a design basis leading to the development of the construction plans and specifications. The objective of the Westminster, East Garden Grove Feasibility Study is to investigate alternatives for flood risk reduction to the Orange County Community.

2 Project Reports/Documents Review

There have been numerous geotechnical reports and drawings that were developed for the system. These were reviewed in depth by Diaz Yourman, GeoPentech, Kinnetic Laboratories (27 Jul 2016) as shown in Appendix G-1. These are summarized in Table 3 of Appendix G-1.

The reports containing Atterberg limit data are summarized in Appendix G1 in Table A-1 and Figure 6b. Using Diaz, Yourman, GeoPentech, and Kinnetic Laboratories (DYGK) numbering, Atterberg limit data can be found in reports 1, 2,5,6, 9, 10, 11, 13, 15, 16, 19, 26, 28, 47, 50, 52, 53, 54, 55, 56, 57, and 58. Reviewing these data show Atterberg limits ranging such that the materials classify anywhere between low plasticity silts and clays to high plasticity silts and clays as well as organic clays. Despite the large number of Atterberg limit tests DYGK identify that Atterberg limits are missing for C06 and most of C04 and C05 east of Goldenrod. Therefore additional investigations will be required during the design phase.

As with Atterberg limit data, most of the investigations of the levees omit C06 and most of C04 and C05 west of Goldenrod. Data for the levees available appear similar to the section shown in Figure 9 of Appendix G-3, which shows the levees constructed of silt, silty clay and lean clay. Based on the engineering as-built drawings which include borings through the centerline of the channel completed prior to construction, the levees appear to have been built to some engineering standards. However, the specifications and foundation reports have not been located.

The repairs as described in Drawing C02-101-SM-Fac, Sep 1994; Drawing C04-101-21M, 11Jul 1996; Drawing C05-101-6R Sep 1993, and Drawing C05-101-14A Jun 1971 appear to be mainly addition of rip-rap to the channel slopes to address sloughing.

2.1 Similar Projects Lessons Learned

Various ditches such as trapezoidal earth, concrete lined, as well as concrete box channels have been constructed as part of the overall system. Construction of the present system was begun in the 1950s and has continued through the present. Projects constructed after the year 2000 are considered “with project,” and projects completed prior to 2000 are considered “without project.” A summary of the constructed projects is included in Table G-1.

Drawing	End	Reach	Type	Scope
C02-101-1A-Fac	7/1/1956	C02	Plans	trapezoidal channel
C02-101-2A	6/16/1959	C02	As-Built	trapezoidal channel
C02-101-4A	9/25/1985	C02	As-Built	trapezoidal channel

Drawing	End	Reach	Type	Scope
C02-101-SM-Fac	9/1/1994	C02	Plans	emergency rock slope protection
C04-101-1A	8/1/1953	C04	As-Built	trapezoidal channel
C04-101-3A	1/1/1954	C04	As-Built	trapezoidal channel
C04-101-5A	1/1/1960	C04	As-Built	trapezoidal channel
C04-101-6A	7/1/1960	C04	As-Built	trapezoidal channel
C04-101-7M	5/1/1961	C04	As-Built	trapezoidal channel
C04-101-8A	12/1/1962	C04	As-Built	trapezoidal channel
C04-101-9A	6/1/1963	C04	As-Built	trapezoidal channel
C04-101-11A	9/1/1964	C04	As-Built	trapezoidal channel
C04-101-12A	4/1/1967	C04	As-Built	trapezoidal channel
C04-101-13A	3/1/1972	C04	As-Built	box channel
C04-101-14A	1/8/1974	C04	As-Built	box channel
C04-101-15A	6/1/1977	C04	As-Built	box channel
C04-101-16A	4/7/1989	C04	As-Built	culvert pipes
C04-101-17	2/1/1991	C04	Plans	box and culvert pipes
C04-101-17R	7/9/1992	C04	Record	box and culvert pipes
C04-701-4R	4/28/1993	C04	Record	box channel
C04-101-18M	3/28/1995	C04	Record	trapezoidal channel
C04-101-19	6/4/1996	C04	Plans	trapezoidal channel
C04-101-21M	7/11/1996	C04	Record	emergency rock slope protection
C04-101-22	4/8/2010	C04	Plans	box channel
C05-101-1A	9/1/1959	C05	As-Built	trapezoidal channel
C05-101-2A	4/1/1960	C05	Plans	trapezoidal channel
C05-101-2A	4/1/1960	C05	Plans	trapezoidal channel
C05-101-3A	8/1/1963	C05	As-Built	trapezoidal channel,
C05-101-4A	12/1/1964	C05	As-Built	trapezoidal channel
C05-101-14A	6/1/1971	C05	As-Built	slope repair
C05-101-5	4/1/1987	C05	Plans	culvert pipes
C05-101-6R	11/1/1990	C05	Plans	trapezoidal channel and box culvert
C05-101-7	9/1/1993	C05	Plans	rip rap lining one side
C05-101-8	9/1/1995	C05	Plans	box channel
C05-101-9	11/14/1995	C05	Plans	box channel
C05-101-8R	9/2/1996	C05	Record	box channel
C05-101-10	4/1/1998	C05	Plans	box channel
C05-101-10R	1/12/1999	C05	Record	box channel
C05-101-13R	2/8/2008	C05	Record	sheetpile

Drawing	End	Reach	Type	Scope
C05-101-14	4/1/2010	C05	Plans	double sheetpile
C05-101-12	6/1/2012	C05	Plans	double sheetpile
C05-101-11	2/15/2002	C05/C06	Record	box channel
C06-101-1A	9/1/1959	C06	As-Built	trapezoidal channel
C06-101-2A	4/1/1960	C06	As-Built	trapezoidal channel
C06-101-3A	8/1/1963	C06	As-Built	trapezoidal channel
C06-101-4A	5/1/1964	C06	As-Built	trapezoidal channel
C06-101-6A	11/1/1968	C06	As-Built	trapezoidal channel and box culvert
C06-101-7A	1/1/1984	C06	As-Built	box channel
C06-701-1	5/31/1985	C06	Plans	trapezoidal channel and box culvert

Table G-1 Summary of constructed projects in the channel system

2.2 Soil Survey Reports for Project Area

Soil survey reports for the project area were surveyed as shown in Figure 5a of Appendix G-1. The surficial soils consist primarily of Young Alluvial Fan Depostis (Qyf), which consist of unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issue from a confined valley or canyon. To a lesser extent, there are deposits of Young Lacustrine, Playa, and Estuarine (Paralic) Deposits (Qyl), which consist of unconsolidated to slightly consolidated, undissected to slightly dissected fine-grain sand, silt, mud, and lcay from lake, playa, and estuarine desposites of various types. Closer to the ocean, to the north of C05, there is a deposit of Old Lacustrine, Playa, and Estuarine (Paralic Deposits) (Qol), which consist of slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types.

Of concern to construction, maps of peat found along channel alignment is provide in Figure 7 or Appendix G-1.

2.3 Local/Regional Geology Reports from State Geologic Survey

Local/Regional geology reports from the state geologic survey were reviewed as shown in Appendix G-1. Potentially liquefiable soils have been mapped and identified for the proposed channels as shown in Appendix G-1, Figure 10, seismic hazard zones have been identified in Appendix G-1, Figure 11a. Faults in the project area are mapped in Figure 11b, and a tsunami inundation map is located in Figure 11c.

3 Applicable Design Guidance

The for each anticipated design feature, applicable design guidance and recommended factors of safety are provided in Table 2.

Table 2. Design features, guidance, and recommended factors of safety

Design Feature	Guidance	Applicable Factor of Safety
Levee/Floodwall	Seepage Control (EM 1110-2-1913 Design and Construction of Levees, EM 1110-2-1901, EM 1110-2-1914)	Seepage berms (FS > 2.8), exit gradients < 0.5 on landside toe
	Slope Stability (EM 1110-2-1913 Design and Construction of Levees)	FS > 1.3 (end of construction), 1.4 (Long-Term Steady Seepage), Rapid Drawdown (1.0 short term loading, 1.2 long term loading)
	Settlement (EM 1110-2-1913 Design and Construction of Levees, EM 1110-1-1904)	Settlement should be such that required level of protection is met after settlement
	Earthquake (ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects, ER 1110-2-1150 Engineering and Design for Civil Works)	Develop a Site-specific PSHA/DSHA. This should include a Response Spectrum Analysis or a Time History Analysis
Floodwall	EM 1110-2-2502 (Retaining and Floodwalls)	Sliding (see Table 4-3), Bearing capacity (FS > 1.5), Overall slope stability (FS > 1.5), Internal stability (FS > 1.5)

4 Additional Subsurface Data Acquisition

No additional subsurface data has been acquired as part of this work. However, extensive geotechnical investigations have been performed by others as summarized in Appendix G-1. Additionally, data gaps have been identified Section 8-10 of Appendix G-1. Because this project resides outside the Chicago District, and because new investigations were not performed as part of this work, subsurface data were not entered into the Chicago District Borehole Database.

5 Alternatives Development

Alternatives were developed by the Los Angeles District, the local sponsor, as well as the Planning Branch of the Chicago District. The tentatively selected plan is the Hybrid plan as shown in Figure G-1. This plan entails alteration of some channel geometry, lining portions of the channels with concrete, and construction of floodwall.

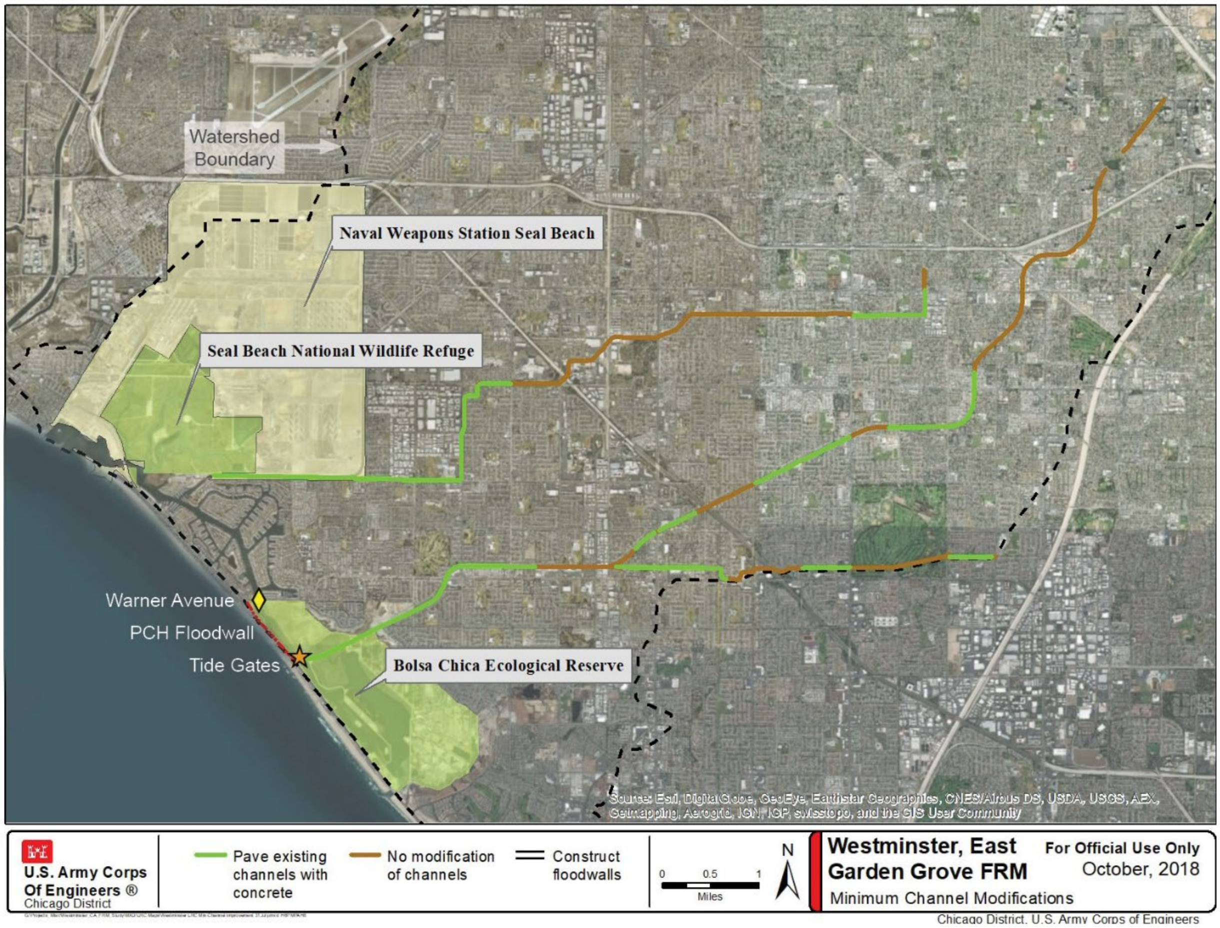

Figure G-1: Minimum Channel Modifications Plan.

The geotechnical section was tasked with evaluating the existing system for fragility. Additionally, upon review of the existing plans, the geotechnical section proposed a tunnel alternative, which is describe in Appendix G-2.

5.1 Alternatives Evaluation

For the proposed alternative, settlement, seepage, stability, and liquefaction analyses are recommended. Seepage and slope stability calculations were performed for the overflow section (Appendix G-6). Additionally seepage and rapid drawdown analyses were evaluated as part of the Fragility Analysis (Appendix G-3). However, additional analyses will be performed during the design phase.

5.2 Geotechnical/Geologic Risk Factors

A risk register was developed based on template provided by the Los Angeles District and is provided in Appendix G-4. The major addition to the risk register from the Chicago District was to include the potential for liquefaction with mitigating measures as a site specific seismic analysis and design based on the site specific seismic analysis. Earthquake accelerations were taken from the ASCE 7 toolbox and are provided in Appendix G-7.

One concern identified during the comment review phase was the issue of impact of concrete lined channels on groundwater recharge. Water reclamation and recharge are not primary goals under the Corps' FRM mission area. However, the Recommended plan avoids paving existing soft bottom channels compared to other study alternatives (including the NED Plan), particularly on the downstream end of the channels, that are more often ponded with water. Further, the recommended plan includes mitigation measures to account for impacts to waters of the U.S., including conversion of soft bottom channels to hard bottom, according to Section 10 of the Rivers and Harbors Act of 1899.

Additionally, based on preliminary analysis, it does not appear that the impact of lining the channels with concrete will significantly affect the amount of recharge to the aquifer below the project. This is because the majority of infiltration is expected to occur through the relatively permeable alluvial surface soils compared to the channels. The drainage area for C04 and C05/C06 channels is 10.9 and 28 square miles, respectively. For these areas, 30% of the area is assumed to be pervious. This represents an area of 11.7 square miles. By contrast, the areas of the channels are 0.12, 0.12, and 0.04 square miles for C02, C05, and C06 respectively for a total of 0.28 square miles. Because the channels, particularly the upstream portions are often dry and because the channels only constitute approximately 2% of the available recharge area, paving the channels will likely have little effect on recharge. However, during the design phase a cost/benefit analysis for using permeable pavement to line the bottom of some or all the proposed channels will be considered, as will other alternatives to increase infiltration. Though the cost of permeable pavement may be 25% more than conventional pavement, this cost may be offset by increased design life or water savings, which will be evaluated within the authority of the project.

5.3 Quantity Computations

The computation of quantities was performed by the Civil Design and Cost Section for this project.

5.4 Geotechnical Analyses

Computations for seepage gradients were performed as part of the fragility analysis (Appendix G-3) and are being checked as part of this review process. Seepage and slope stability calculations were performed for the overflow section (Appendix G-6).

5.5 Borrow Materials and Construction Considerations

Borrow materials and construction considerations will be added during the design phase of the project.

5.6 Level of Detail Sufficient to Support a Reasonable Cost Estimate

Construction considerations such as groundwater depth, corrosive soils, problematic soils such as peats, adjacent faults, surveys of past soil tests and investigations are provided in Appendix G-1. Pile driving conditions are described the Civil Design Appendix. These investigations are suitable to identify potential construction concerns and are of sufficient detail to provide a reasonable cost estimate.

5.7 Subsurface Data on Drawings

There are a significant number of borings. Geotechnical cross sections and parameter estimations for these sections is included in Appendix G5. During the design phase geotechnical investigations will be added to the design drawings.

5.8 Design Details on Drawings

The geotechnical sections have been developed and are shown in Appendix G5. Additional details on the design drawings be be include as part of the design phase.

5.9 Overall Report and Drawings Integration

The overall report and drawings have been reviewed for proper integration.

6 Appendices

Appendix G-1	Westminster, East Garden Grove, California Flood Risk Management Feasibility Study: Preliminary Geotechnical Appendix Orange County, California. July 27, 2016 Prepared for U.S. Army Corps of Engineers P.O. Box 532711 Los Angeles, CA 90053-2325 Diaz•Yourman – GeoPentech – Kinnetic Laboratories / Joint Venture 1616 E. 17th Street Santa Ana, CA 92705-8509 (714) 245-2920 DY Project No. 2011-026.11, GP Project No. 09021D
Appendix G-2	Geotechnical and Design Considerations for Tunnel Alternative
Appendix G-3	Fragility Analysis without Project
Appendix G-4	Geotechnical Risk Register
Appendix G-5	Geotechnical cross sections by subsection along channel centerlines
Appendix G-6	Overflow area analysis
Appendix G-7	ASCE Design Hazards Reports for Tetra Tech Sites 01 – 05

WESTMINSTER, EAST GARDEN GROVE, CALIFORNIA FLOOD RISK MANAGEMENT FEASIBILITY STUDY: PRELIMINARY GEOTECHNICAL APPENDIX ORANGE COUNTY, CALIFORNIA

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Westminster, East Garden Grove, California Flood Risk Management Feasibility Study: Preliminary Geotechnical Appendix Orange County, California

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LIST OF ACRONYMS AND ABBREVIATIONS

ASCE	American Society of Civil Engineers
bgs	below ground surface
CDMG	California Division of Mines and Geology
CGS	California Geological Survey
CPT	Cone Penetration Test
OCR	Overconsolidation Ratio
PGA	Peak Ground Acceleration
SPT	Standard Penetration Test
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey

1.0 INTRODUCTION

The Westminster Watershed drainage channels are a major component of the flood control system for northern Orange County, California. As shown on Figure 1a, the system is composed of four channels: C02 Bolsa Chica Channel; C04 Westminster Channel; C05 East Garden Grove Wintersburg Channel; and C06 Ocean View Channel. The U.S. Army Corps of Engineers (USACE) has completed hydraulic analyses of these channels and found the system is under-designed, largely due to urban growth since they were constructed (USACE, 2007). Most reaches cannot support the 100-yr flood demand, and several reaches will break[1]out in the 25-yr event (USACE, 2007). As a result, the USACE has initiated a feasibility study to evaluate alternatives for potential system modifications and upgrades.

This preliminary geotechnical appendix presents a summary of the available data along the alignments relating to geotechnical and geologic conditions. The collected and reviewed data were provided by the United States Army Corps of Engineers (USACE) and obtained from the United States Geological Survey (USGS) and California Geological Survey (CGS). The reviewed data included reports, maps, existing previous explorations, published literature, aerial photographs, and other relevant in-house reports.

The information contained in this appendix includes a preliminary geotechnical characterization of the subsurface and groundwater conditions along the channel alignments and geologic/seismic hazards, such as fault rupture, liquefaction, lateral spreading, seismically-induced settlement, subsidence, landsliding, tsunami, and erosion. Available information was used to assess the USACE's proposed upgrade alternatives and make recommendations on future geotechnical investigations needed for the design and construction of the proposed alternatives. Data gaps within the available information were also identified along the various reaches of the channels. No field exploration, laboratory testing, or engineering analyses were performed as part of this review.

[1] The term "breakout" can refer either to levee overtopping or, if the channel is sunk into the ground, then breakout can describe water overtopping the channel banks (Chicago District, USACE)

2.0 EXISTING CHANNELS AND PROPOSED ALTERNATIVES[2]

Each channel is divided into several reaches, and it is our understanding that the reach numbering system is based on historic nomenclature that is generally not reflective of a specific channel geometry or construction type. However, channel geometry and construction type are typically continuous between street or freeway crossings; therefore, we have divided the channel reaches into "subreaches" at each major bridge/crossing, based on the information provided to us by Mr. Larry Walsh of the USACE during our site visit on April 29, 2016. These subreaches are shown on Figure 1a. Figure 1b shows the existing channel geometry and construction type. Existing channel configurations range from rectangular concrete-lined (box) to vertical sheet piles with natural inverts, but most reaches are trapezoidal with concrete, natural, or rip-rap sidewalls and concrete or natural inverts. Figure 2 shows the existing crossings. Most crossings are bridges with piers or flared wingwall boxes.

The existing channels are located in densely populated areas, with most reaches developed adjacent to existing structures, including residential units, and in areas with shallow groundwater. Therefore, physical constraints on the width and depth of the channels limit upgrade options.

Previous studies by the USACE identified four alternative system upgrades, considering right-of-way limits and groundwater depth constraints, referred to as Alternatives 2, 3, 4, and 5. Alternative 1 represents “no channel upgrades” and is used as the base case for comparison to Alternatives 2 through 5 as part of the USACE’s flood risk management study. Alternatives 2 through 5 are shown side-by-side on Figure 3a and individually in more detail on Figures 3b through 3e. Table 1 summarizes the general characteristics of the proposed alternatives. Further discussion on each alternative is provided below.

In general, most of the alternatives involve converting trapezoidal channels with natural inverts and natural or rip-rap sidewalls to either concrete trapezoidal or rectangular channels. As shown on Figure 3a, Alternatives 2 and 5 are similar, generally consisting of converting most reaches to concrete trapezoidal channels. Alternative 2 represents a minimal upgrade effort, which we understand will still leave the channel under-designed, as many reaches will continue to breakout in short return period flood events (i.e., the 50-yr or 75-yr). Alternative 5 improves the C05/C06 system capacity by connecting the two channels via a new-construction buried conduit under Ward Street and retaining stormwater in a new-construction detention basin in Mile Square Park.

[2] After development of the Geotechnical Feasibility Appendix by DYGK, the alternatives were updated by the Chicago District. See the main text "Development of Alternative Plans" section 2.18 for current description of the alternatives considered.

It is our understanding that the configuration of the basin is conceptual at this time, but the USACE has found that shallow groundwater and higher ground surface elevations along the northern side of the park would limit the capacity of the basin.

Alternative 3, as shown on Figures 3a and 3c, consists of converting most reaches to concrete rectangular channels with three-foot-high above-grade floodwalls or adding new three-foot-high floodwalls to existing channel reaches. A few reaches in the C05/C06 system will also be converted to concrete rectangular channels without floodwalls.

Figures 3a and 3d show Alternative 4, which is the most extensive upgrade alternative. Alternative 4 entails converting most of the eastern reaches into concrete rectangular channels, as well as adding three-foot-high floodwalls in some reaches of the C04 channel. Along the coast, Alternative 4 will require replacing the existing channel with a concrete rectangular channel with natural invert for part of the C02 system and extending the concrete rectangular channel with sheet pile walls inland for part of the C05 system.

Significant system-wide bridge upgrades will also be required to increase channel capacities, as shown on Figure 4. These modifications are generalized and independent of the alternative; however the actual geometries of the modifications will be consistent with the hydraulic baseline conditions of the selected alternative.

3.0 AVAILABLE DATA SOURCES

The USACE provided geospatial data in the form of ESRI ArcGIS shapefiles and Google Earth KMZ files and tabulated draft reach notes with information on existing channel conditions and proposed alternatives. These data were used to develop Figures 1 through 4, which show the existing conditions and proposed alternatives.

In addition, the USACE also provided numerous previously completed geotechnical reports and plan sets for various sections of the channel systems. These reports and plans are listed in Table 2. We also supplemented Table 2 with available in-house reports that are within the project area.

Publicly available datasets applicable to the area of interest, such as State regulatory maps and reports for seismic hazard zones, as well as geologic and topographic maps, were also reviewed and used in our study as appropriate.

4.0 GEOLOGIC SETTING

The channel systems are located within the relatively flat coastal plain of northwestern Orange County, and most of the project area is within the prehistoric Santa Ana River floodplain. The elevation ranges from sea level to roughly El. +100 feet.

The distribution of Quaternary-aged surficial deposits within the project area is shown on Figure 5a, and the legend is included on Figure 5b. As observed on Figure 5a, most of the project area is covered by young alluvial fan deposits (Qyf), which are described as unconsolidated to slightly consolidated, undissected to slightly dissected silt- to boulder-sized deposits emanating from a confined valley, Late Pleistocene to Holocene in age. Dissected remnants of young lacustrine, playa, and estuarine deposits (Qyl) and young alluvial valley deposits (Qya) are also dispersed throughout the project area. Near the coast, dissected mesas composed of old lacustrine, playa, and estuarine deposits (Qol) remain slightly elevated relative to the rest of the coastal plain. The western reaches of the C02 channel are in artificial fill in Huntington Harbor. Details on the placement of the fill (i.e., engineered, hydraulically placed, etc.) is unknown.

Table 3 lists the mapped surficial geologic units found within one mile of each reach. (One mile is judged to be sufficient to capture map scale errors as well as shallow, below grade materials that may potentially be encountered along the reaches.) In general, the Quaternary geology map shows most of the surficial deposits in the project area are characterized as loose to medium dense granular materials within interspersed cohesive sediments. However, based on the subsurface explorations reported in the documents listed in Table 2, most of the shallow materials in the project area appear to be composed of interbedded loose to medium dense sand and silty sand, and soft to stiff silt and clay.

5.0 GROUNDWATER CONDITIONS

Figure 5a shows groundwater contours for the historically highest depth below ground surface for the project area, as reported in California Geological Survey (formerly California Division of Mines and Geology) Seismic Hazard Zone Reports. The groundwater contours are consistent with the groundwater depths identified in the reports listed in Table 2. Note that these contours represent shallowest groundwater levels previously documented (i.e., historically highest), and actual conditions along the alignment may vary at any given time.

Depth to groundwater is generally shallow (within 10 feet bgs) throughout most of the project area. Groundwater is deepest near the northeastern reach of the C05 system (approximately 30 feet bgs). It is noted that most of the region is underlain by a relatively shallow aquitard, and therefore shallow groundwater levels within and above the aquitard can be sensitive to rainfall (e.g., Report No. 13 in Table 2).

6.0 PREVIOUS EXPLORATIONS

6.1 AVAILABLE DOCUMENTS

The USACE provided geotechnical reports and plan sets with subsurface exploration logs of varying vintage and quality. We reviewed the documents listed in Table 2 (which also includes applicable in-house reports) and summarized pertinent geotechnical data in attachment Table A-1 at the end of this appendix. The existing reports are identified by a “Report No.” on both Table 2 and Table A-1. Figure 6a shows the reports associated with each subreach, also identified by the “Report No.”

As shown on Figure 6a, some geotechnical information is available for most reaches of the channel systems. However, the information varies in both quality and vintage; for example, recent site-specific studies completed for channel improvements in the coastal reaches of C05 are generally more thorough and consistent with the current geotechnical state of practice; datasets for the eastern reaches consist mostly of incomplete boring logs from the 1950s.

Key geotechnical information of interest for the evaluation of the various alternatives in feasibility-level studies would typically be obtained from borings, test pits, and Cone Penetration Tests (CPT), along with laboratory tests to characterize the soils. Accordingly, we focused our review of the available information on identifying, examining, and summarizing the following items:

- **Field Tests:** Blow counts, CPTs
- **Classification of Soils and Index Properties Tests:** moisture/density, gradation, Atterberg limits
- **Strength Tests:** Unconfined compression, direct shear, and unconfined undrained triaxial
- **Compressibility Tests:** Consolidation
- **Corrosion Tests:** Soil corrosivity to concrete and metals

The distribution of available field and laboratory datasets along the channel systems is shown on Figures 6b through 6e. Figure 6b shows the distribution of subreaches with field testing for CPTs and soil blow counts. Blow count information was obtained from Standard Penetration Test (SPT) samplers, California samplers, and various other samplers, as reported on the boring logs reviewed for this study. Figure 6b also shows locations with laboratory tests for moisture content/density, gradation, and Atterberg limits. Figure 6c shows the distribution of strength tests (unconfined compression, direct shear, and unconfined undrained triaxial) and compressibility tests (consolidation). Figures 6d and 6e show the distribution of soil corrosivity tests for concrete and metals, respectively. The reports containing these datasets are shown on Figure 6a and summarized in Table A-1. Criteria for corrosivity tests and descriptions are generally consistent with the State of California Department of Transportation (Caltrans) Standard Test Methods.

Based on a review of the boring logs, CPT signatures, and laboratory tests, the shallow stratigraphy (~50 feet bgs) is highly variable both laterally and vertically, even within channel system reaches. Although the details vary depending on exact location, most of the borings drilled and logged indicate packages of loose sand and silty sand interbedded with denser sandy material, soft to stiff clays or silts with and without organics, and locally soft peat at varying depths. The key geotechnical considerations for these types of materials and this region include but are not limited to:

- **Strength and compressibility:** poor subsurface materials due to the presence of peat; non-peat organics; and soft soils.
- **Seismic hazards:** strong ground motions, potential for liquefaction, seismically-induced settlements, seismically-induced lateral spreading, and faulting.

To illustrate the spatial distribution of soil conditions that are prone to these geotechnical issues, a series of figures have been assembled based on the reviewed data summarized in Table A-1. Specifically, soil conditions related to strength and compressibility due to the presence of peat, non-peat organic material, and soft soils are shown on Figures 7, 8, and 9, respectively. Seismic hazards due to liquefaction and faulting are highlighted in Figures 10 and 11a.

Peat and soft fine-grained soils are common throughout the project area. Figure 7 shows the distribution of peat along the channel systems based on the available previous studies. As shown on Figure 7, peat is reported at shallow depths throughout most of the C05/C06 system, as well as at buried depths of up to 40 feet bgs. Based on our review of the previous studies, peat was typically reported in thicknesses of up to 5 feet (locally up to 10 feet) and commonly interbedded with loose to dense sand and silty sand, and soft to stiff clays or silts with varying organic content. With respect to geotechnical considerations, shallow peat is typically highly compressible with low shear strength. Subsidence can also occur as surficial peat deposits decompose due to oxidation.

Soft soils with some organic content (but not considered peat) were also identified throughout the channel systems, as shown on Figure 8. It is noted that the distinction between peat and organic soils is based on how the soils were logged by others; therefore, it is possible that some peat and non-peat organic soils could have similar organic contents. The data reviewed and summarized in Table A-1 show soft organic material was only reported in the C05/C06 system, roughly coincident with the distribution of peat. Based on the data available, the organic soils are interbedded with other fine and coarse-grained soils. As with peat, soft organic soils are compressible and typically have low shear strength.

Figure 9 shows the spatial distribution of non-organic soft clays and silts reported throughout the channel systems. In general, soils were identified as soft in our review based on a combination of CPT log signatures and/or boring log descriptions or blow counts. As shown on Figure 9, shallow soft clay and silt is common throughout the project area, and also common at depths greater than 25 feet bgs within the western half of the C05/C06 system. The thickness of the soft units varies considerably throughout the channel systems from less than 5 feet to greater than 20 feet. The soft clay and silt is usually interbedded with loose to medium dense sand and silty sand, peat or organic soils, or stiffer clays or silts. With respect to geotechnical properties, soft clay and silt is compressible and has low strength.

Figure 10 shows the spatial distribution of soils potentially subject to liquefaction based on the information obtained from the existing reports. These site-specific studies are consistent with the State regulatory maps shown on Figure 11a that identify potentially liquefiable zones. Potentially liquefiable soils are of concern from a geotechnical standpoint, as the soils are subject to loss of strength and potentially significant ground deformations during earthquake shaking (see Section 7.3 for further discussion). Based on our review of the CPTs and borings drilled and logged by others, it appears that liquefiable, shallow (i.e., within 40 or 50 feet bgs), loose sands and silty sands that are below the historically highest reported groundwater depths occur or potentially occur along all reaches of the channel systems. These liquefiable sediments are often interbedded with non-liquefiable soils, such as denser sandy material, peat, or soft to stiff clays or silts.

6.2 DATA GAPS

Field data, typically in the form of soil blow counts or CPT measurements, are usually necessary for the evaluation of the type of work proposed by the USACE. Laboratory data including index testing (gradation, moisture/density, Atterberg Limits, etc.), strength testing (unconfined compression, unconfined undrained triaxial, etc.), and consolidation are also typically needed.

Figures 6b and 6c show the subreaches within the channel systems where geotechnical data are available in blue; the data gaps are shown in white. In general, few subreaches east of Goldenwest Street have index testing information, as shown on Figure 6b (although moisture/density tests are available along most of the C05 channel). While there appears to be abundant spatial coverage of soil blow counts (with the exception of the C06 channel and the eastern half of the C04 channel; see Figure 6b), it is noted that many of the borings were shallow, on the order of 20 feet in depth. As shown on Figure 6c, few subreaches within the channel systems contain soil strength testing information. Although direct shear tests were performed at many subreaches west of Interstate 405, unconfined compression and unconsolidated undrained triaxial test data are only available in the western reaches of the C05 channel and at two locations near Interstate 405. Consolidation test information is only available for the western reaches of the C05 channel and most western reaches of the C04 channel.

In summary, the following geotechnical data gaps were identified throughout the channel systems:

- Soil blow count data are not available in most of the C06 channel and the eastern reaches of the C04 channel. Soil blow count data that are available may not extend to sufficient depths.
- CPT data are not available in most of the C06 channel and the eastern reaches of the C04 and C05 channels.
- Moisture/density data are not available in most of the C06 channel and the eastern reaches of the C04 channel.
- Gradation data and Atterberg Limits are not available in the C06 channel and most of the C04 and C05 channels east of Goldenwest Street.
- Direct shear test data are not available in the C06 channel and most of the C04 and C05 channels east of Goldenwest Street.
- Consolidation, unconfined compression, and unconsolidated undrained triaxial test data are not available in the C06 channel and most of the C04, C05, and C06 channels.

7.0 GEOLOGICAL AND SESMIC HAZARDS IMPACT

7.1 GROUND MOTIONS

The channel systems are located within a seismically active region of southern California, as evidenced by Quaternary faulting and historic earthquakes. The Peak Ground Acceleration (PGA) for a 2% probability of exceedance in 50 years (equivalent to the 2,475-yr average return period), based on the 2014 National Seismic Hazard Maps (Petersen et al., 2014) exceeds 0.5 g for most of the project area at this hazard level. Similarly, the PGA for a 10% probability of exceedance in 50 years (equivalent to the 475-yr average return period) exceeds 0.25 g. (It is noted that the 2014 National Seismic Hazard Maps are based on a V_{S30} of 760 m/s and site-specific PSHAs using site-specific V_{S30} values would produce different accelerations; nevertheless, these maps provide an idea of the relative ground motion hazard in the project area.)

7.2 FAULT RUPTURE HAZARD

As shown on Figure 11a, the western-most reaches of the channel systems are within the Newport-Inglewood Alquist-Priolo Earthquake Fault Zone (CDMG, 1986a,b; CGS, 2002). The Alquist-Priolo Act prohibits the construction of habitable structures across active faults and within 50 feet of an active fault (Bryant and Hart, 2007); however, as the channels are not habitable, they are not subject to compliance with the Alquist-Priolo Act. Nevertheless, local governing agencies, such as cities or counties, may have additional requirements for construction or retrofit of non-habitable structures that cross known active faults. Note that specific guidance or criteria for evaluating fault crossings of open channels is very limited, though some guidance may be gathered from treatment of analogous structures such as pipelines, levees, or dams. In general, it is expected that the design conditions for an intermittently flowing open channel will be somewhat less stringent than for these other types of infrastructure.

The Newport-Inglewood Fault Zone is capable of producing surface-rupturing earthquakes with average lateral displacements on the order of 2 meters (6.5 feet) and local maximum displacements of up to 4 meters (13 feet) (Leonard, 2010, 2012; Wells and Coppersmith, 1994). Stepovers across multiple fault splays or strands and changes in fault strike or orientation can create localized transpression and transtension, resulting in local vertical displacements of a couple feet. Lateral surface displacements can reduce the cross-sectional area of the channels and are therefore a potential flood breach hazard. Vertical surface displacements can disrupt channel gradients, also serving as a potential flood hazard.

Other mapped but unnamed pre-Quaternary faults are within the project area, east of Interstate 405 (Jennings and Bryant, 2010), as shown on Figure 11b. These faults are not known to be active and therefore do not present a surface rupture hazard.

7.3 LIQUEFACTION, LATERAL SPREADING, AND SEISMICALLY-INDUCED SETTLEMENT

Liquefaction occurs when relatively loose, saturated, non-cohesive soils undergo a temporary loss of stiffness and strength during strong ground shaking. As a consequence, permanent ground and surface deformation can occur. Liquefaction potential is greatest where the groundwater level is shallow, and submerged, loose, fine sands occur within a depth of 40 to 50 feet or less below the ground surface (Martin and Lew, 1999). Liquefaction potential tends to decrease as grain size increases and as clay and gravel content increase. Higher ground accelerations and shaking durations during earthquakes increases the liquefaction potential.

Lateral spreading occurs when soils liquefy and slide or flow downhill, or breach an open slope face, resulting in permanent ground deformation. Thus, open slope faces composed of materials susceptible to liquefaction are also potentially susceptible to lateral spreading. Seismically-induced settlement can occur when unsaturated loose to medium-dense granular soils are densified during ground shaking. Therefore, soils susceptible to liquefaction are also often susceptible to seismically-induced settlement.

Based on a review of the State Regulatory Maps, which include zones of required investigation for liquefaction and landslide hazards per the Seismic Hazards Mapping Act of 1990 (cf. CGS, 2002), all existing and proposed reaches of the channel systems are within Liquefaction Zones of Required Investigation, as shown on Figure 11. Nearly all of the site-specific reports reviewed herein also identified liquefaction as a hazard, as shown on Figure 7 and discussed in Section 6.

Several recent reports for coastal reaches of the channel systems included quantitative analysis of lateral spreading deformation and seismically-induced settlements based on several methods; i.e., some reports utilized “CLiq” or similar software for CPT liquefaction analysis (which is generally suitable for a screening-level effort but often insufficient for design), and others used more sophisticated methods. Developing updated quantitative estimates of lateral spreading deformations or seismically-induced settlements is beyond the scope of this report; however, it can be assumed that the open-face channel locations subject to liquefaction may be subject to lateral spreading (perhaps on the order of a few feet), and areas subject to liquefaction may also be subject to seismically-induced settlement (perhaps on the order of several inches to one foot). Site-specific analyses should be performed if this level of risk is unacceptable for the project.

7.4 CYCLIC SOFTENING

Cyclic softening refers to the phenomenon of cyclic loading of clays and plastic silts producing rapidly increasing strains and loss of strength in relatively cohesive materials. This effect is less common than the analogous loss of strength in sands, but notable failures related to cyclic softening have occurred, such as the Alaskan Fourth Avenue Slide in 1964 or foundation failures during the 1999 Chi Chi Earthquake (Idriss and Boulanger, 2008). Soils in the project area may be susceptible to cyclic softening when the earthquake shaking levels are high, and soft, saturated, clayey soils are present within the upper portions of the subsurface profile. Typically, the potential for cyclic softening will tend to increase as the undrained strength and/or overconsolidation ratio (OCR) of a clayey soil decreases.

Based on our data review, much of the project area is underlain by peat, organic soils, and soft clays and silts which may be susceptible to cyclic softening. In particular, peat and organic soils generally have very low undrained strengths and would be potentially susceptible. Previous investigations did not systematically evaluate cyclic softening of clayey deposits. Due to the presence of potentially susceptible materials throughout the project area, further evaluations should be conducted. Potential consequences of cyclic softening include deformation of channel walls and inverts, bridge and structure foundation movements, and localized landsliding or lateral spreading into the channels. Developing quantitative estimates of cyclic softening is not possible given the limited amount of data available and is beyond the scope of this report. However, it can be assumed that areas susceptible to cyclic softening could be subject to deformations similar in magnitude to those experienced by areas subject to liquefaction.

7.5 REGIONAL SUBSIDENCE

Ground surface subsidence and fissuring generally occurs when the extraction of fluids or gas from the subsurface results in a gradual lowering and flexure of the overlying ground surface on a regional scale. The western reaches of the channel systems are within the active Huntington

Beach oilfield. However, subsidence due to fluid extraction is closely monitored. Therefore, the potential for regional subsidence due to fluid extraction is considered low.

Importantly, we note localized subsidence or sinkholes related to settlement of shallow, loose or soft, saturated soils during dewatering or structural loading is possible and has been observed in the project area (e.g., at Slater Pump Station; see Report No. 17 in Table 2).

7.6 LANDSLIDE

Landslide hazards are generally highest in areas of moderate to steep terrain that are underlain by unfavorably oriented geologic discontinuities. The channel systems are located on flat terrain, and the reaches are not within Earthquake-Induced Landslide Zones of Required Investigations. Therefore, the potential for landslide hazards is considered remote.

7.7 TSUNAMI

A tsunami is a sea wave generated by a large submarine landslide or an earthquake-related ground deformation beneath the ocean. Historic tsunamis have been observed to produce a run-up onshore of several tens of feet in extreme cases. As the channel systems are hydraulically connected to the ocean and locally highly influenced by tides, the western reaches of the channel systems are also susceptible to tsunami-induced flooding. Figure 11c shows the tsunami inundation map for the project area (State of California, 2009).

7.8 EROSION

Most of the existing reaches of the channel systems are currently constructed of natural inverts and natural or rip-rap side slopes. Therefore, surficial sloughing or erosion is possible considering the existing channel conditions. The alternative modifications developed by the USACE entail converting most reaches to concrete-lined side slopes, which will eliminate channel wall erosion in those reaches. Note that generally, coarser-grained material will tend to be more erodible than finer-grained soils. Erosion susceptibility of natural inverts is also based on hydraulic factors (such as sediment loading, channel capacity, velocity, etc.) and is beyond the scope of this report; however, scour analyses should be completed to ensure natural inverts and bridge supports are protected.

7.9 METHANE AND HYDROGEN SULFIDE GASES

The western reaches of the channel systems are within the active Huntington Beach oilfield. Oil and gas extraction are active in the area, and methane and hydrogen sulfide gases may be a hazard during construction. We recommend that prior to design and construction in excavation areas, a specialist be consulted and a site-specific study be conducted to evaluate the potential occurrence and impact of methane and hydrogen sulfide gases. Decomposition of organic material, such as peat under anaerobic conditions, can also result in methane accumulation within groundwater-saturated soils[3]. Therefore, methane occurrence should be evaluated for excavations in areas where peat or high organic content soils are present. We also note that undocumented well casings from oil and gas extraction could be encountered during construction.

[3] Gas in the subsurface could contribute to under-consolidation, which could potentially increase settlement. (Chicago District, USACE)

8.0 GEOTECHNICAL RECOMMENDATIONS

8.1 GENERAL

The USACE is evaluating Alternatives 2, 3, 4, and 5 to upgrade existing flood control channels by increasing the capacity of the channels. The alternatives generally involve converting channel geometries from trapezoidal to rectangular, converting construction type from natural or rip-rap to concrete, or adding floodwalls. Specifically, Alternative 2 will generally convert trapezoidal channels with natural or rip-rap side slopes to concrete trapezoidal channels. Alternative 3 generally entails converting the trapezoidal channels to rectangular concrete channels, with the addition of 3-foot-high floodwalls in some locations to the existing or modified rectangular channels. Alternative 4 involves conversions similar to Alternative 3, plus consideration for new sheet pile walls for coastal reaches of the rectangular conversions. Alternative 5 entails the conversion of trapezoidal channels with natural or rip-rap side slopes to concrete trapezoidal channels and the construction of a new detention basin at Mile Square Park along the C06 channel and a new buried conduit connection to the C05 channel.

USACE's evaluation of the various alternatives has considered that the channels are located in densely populated areas, adjacent to existing structures (including residential units) and within areas of shallow groundwater. Consequently, physical constraints on the width and depth of the channels limit upgrade options. In addition to the physical constraints, most of the channel reaches are located in areas of poor subsurface conditions characterized by variable stratigraphy comprised of loose sand and silty sand interbedded with denser sandy material, and the presence of soft to stiff clays or silts with or without organics, and locally soft peat at varying depths. Key geotechnical considerations identified for such subsurface conditions are strength, compressibility, and the potential for liquefaction or cyclic softening.

The following recommendations are based on our review of the available information provided by USACE, published information, and our in-house information. As part of the review process, data gaps were identified along the various sections of the channels as described in Section 6.2. The following recommendations are preliminary and intended for general planning purposes only. Figures 12a through 12d summarize key general conditions and key alternative-specific recommendations relating to geotechnical considerations for the channel systems. Figure 13 summarizes key general conditions and geotechnical recommendations for the proposed bridge upgrades. General construction access constraints are also stated on the figures.

Specific geotechnical design parameters and construction activity details are beyond the scope of this preliminary geotechnical appendix. In general, interim and final geotechnical design parameters and construction methods and measures should be consistent with applicable USACE guidelines and practices, such as the "ASCE Technical Engineering and Design Guides as Adapted from the US Army Corps of Engineers" (ASCE, 1994, 1996a,b).

8.2 CHANNEL FOUNDATIONS AND WALLS BELOW GRADE

In general, foundation support for the proposed rectangular or trapezoidal concrete channel walls can be on continuous footings, with adequate width established on engineered fill. Footings will need to be founded in competent ground. Shallow unsuitable soils throughout most of the project area will require improvement by overexcavation or treatment, as discussed in Section 8.8 and shown for each alternative on Figures 12a through 12d. The thickness of the engineered fill will depend on the overexcavation depths required to remove and either to re-compact or replace unsuitable subgrade material. The foundations would typically be placed below the bottom of the channel slab/mat. Appropriate bearing pressures for foundation design, along with lateral passive resistance and coefficient of friction, should be provided for the footings. Corresponding total and differential settlements due to loading conditions (e.g., the weight of water in the channel and the weight of the structural elements, including additional loading for the three-foot-high floodwalls proposed in Alternatives 3 and 4) should be computed and used in the structural design of the wall footings and channel bottom slab/mat. Depending on the location of the selected design groundwater level, the channel bottom/mat may also be required to be designed to incorporated hydrostatic uplift pressures or be provided with a permanent subsurface drainage system capable of relieving the hydrostatic pressure and discharging the collected water.

Channel walls below grade should be designed to resist lateral earth pressures plus any surcharge from adjacent loads. Channel walls should also be designed to resist hydrostatic pressures (equivalent fluid pressure of 62.4 pounds per cubic foot), or be provided with positive drainage behind the wall. Miradrain drainage panels (or the equivalent) or pea gravel wrapped in filter fabric should be placed behind wall sections. The drain should be connected to a perforated discharge pipe at the base of the wall. The drain pipe should be of adequate capacity to accommodate the anticipated water flow and placed with perforations down along the base of the wall. The filter gravel should meet the requirements of Class 2 Permeable Material as defined in the current State of California, Department of Transportation, Standard Specifications. Alternatively, ¾-inch crushed rock or gravel separated from the on-site soils by an appropriate filter fabric can be used. The crushed rock or gravel should have less than 5% passing a No. 200 sieve. In addition to the earth pressures discussed above, the upper 10 feet of the channel walls below grade adjacent to areas subject to vehicular traffic should be designed to resist a uniform lateral pressure acting as a result of the surcharge behind the walls due to vehicular traffic. Furthermore, the walls below grade should be designed to support an incremental seismic lateral pressure applied uniformly along the wall height in addition to the above-mentioned lateral earth pressures.

Lateral earth pressure surcharges due to adjacent structures located within a 1:1 line projected upwards from the bottom of the channel wall footings should be included. Other surcharge conditions, such as those generated by heavy equipment that would be kept onsite for a significant length of time, should also be addressed on a case-by-case basis.

8.3 BRIDGE/ABUTMENT FOUNDATIONS

Figure 13 shows the distribution of bridges and key geotechnical considerations/recommendations throughout the project area. In general, unsuitable soils (e.g., peat, organic soil, soft clay/silt, loose sand, etc.) are located throughout most of the project area at variable depths and thicknesses, based on the information available. Therefore, deep foundations will likely be required for the bridge foundations. Deep foundation systems should be founded in competent material, such as a firm or dense alluvium, which may be on the order of 40 feet to 60 feet bgs in the project area. Site-specific studies to confirm design foundation depths will be needed. If deep piles are utilized, pile testing programs should be considered. At sites where firm or dense native soil is shallow and relatively lightly-loaded structures are considered, spread footings may be feasible. Spread footings should be founded on engineered fill.

If bridges are subject to seismic performance requirements, site-specific studies will likely be required, and mitigation of seismically-induced displacements, such as liquefaction, cyclic softening, or lateral spreading may be required. Areas within Caltrans jurisdiction should consider Caltrans requirements and/or guidelines, as appropriate.

8.4 SEISMIC DESIGN PARAMETERS

Seismic design parameters may be needed to support the design of bridge abutment upgrades as well as upgrades to the channel systems that require the concrete-lined channels. It is noted that liquefaction along the channel systems is a potential seismic hazard, which would result in a

Site Class F designation per ASCE 7-10. For this reason, ground motions should be developed based on site-specific hazard analyses for outcropping conditions at depth, and the seismic demand at the ground surface should be based on site response analysis to appropriately capture the effects of local site conditions in accordance with Chapter 21 of ASCE 7-10. Areas within Caltrans jurisdiction should consider Caltrans requirements and/or guidelines, as appropriate. Similarly, the results of the site response analysis should be used to carry out liquefaction[4]-triggering evaluations, as well as in the development of any liquefaction remediation measures.

8.5 LIQUEFACTION, CYCLIC SOFTENING, AND RELATED GROUND DEFORMATIONS

Based on the information available, all reaches of the channel systems are potentially susceptible to liquefaction and significant portions are potentially susceptible to cyclic softening, along with related ground deformations. Related ground deformations include seismically-induced settlement, lateral spreading, loss of bearing capacity, increased retaining wall pressures, and others. The deformations associated with these can be on the order of several inches to several feet. Site-specific analyses should be performed to assess the magnitude of deformations and evaluate the level of risk. Note that soils subjected to liquefaction or cyclic softening would lose bearing capacity and the ability to support structural loads associated with most of the proposed channel modifications and bridge abutment modifications.

8.6 FAULT OFFSET

The western reaches of the channel systems (i.e., Reaches 1 and 20) cross the Newport-Inglewood Fault Alquist-Priolo Zone. Although the channels are not subject to the Alquist-Priolo Act (as the Act applies only to human-occupied structures), significant horizontal and vertical offsets within the fault zone during a surface-rupturing earthquake can disrupt channel gradients and induce breakout flooding. If this risk is not acceptable for the project criteria, then additional assessments to better understand and estimate coseismic displacements are recommended. Results of these assessments can be integrated into the selected alternative upgrade or used for emergency repair planning should the system become disrupted as a result of a large earthquake.

8.7 SULFATE ATTACK AND CORROSION POTENTIAL OF SOILS

Figure 6d shows the distribution of areas where soils are corrosive to concrete and areas where no data are available. Soils corrosive to concrete and metals were identified in most of the studies reviewed. For planning purposes, soils in the project area can be assumed to be corrosive to

[4] In addition to liquefaction, cyclic softening will also be considered. (Chicago District)

concrete, and use of cements resistant to sulfate attack can be assumed where concrete linings are planned. It is recommended that additional chemical laboratory corrosivity testing be performed along the channel systems in areas of planned concrete-lined channel upgrades for soluble sulfate content, cations and associated tests, and pH when the final alternative is selected by the USACE.

Similarly, based on the documents reviewed, soils within several reaches of the channel systems are corrosive to metals. Figure 6e shows the distribution of soils corrosive to metals, as well as data gaps where no information on soil corrosivity potential is available. For planning purposes, soils in the project area can be assumed to be corrosive to metals, and a corrosion consultant should be contacted to provide appropriate measures against corrosion (e.g., cathodic protection) to metallic piles, piping, or conduits used in this project. When the final alternative is selected by the USACE, it is recommended that resistivity tests be performed in reaches where metallic materials are planned.

8.8 EXCAVATIONS, BACKFILL, AND TEMPORARY SHORING

Based on the information available, potentially unsuitable soils (e.g., peat, organic soil, soft clay/silt, loose sand, etc.) are located throughout the project area at variable depths and thicknesses. Channel and bridge foundations should be founded on competent materials; accordingly, unsuitable materials will need to be overexcavated or treated. Figures 12a through 12d identify areas of expected overexcavation (as well as data gaps) for each alternative, based on the information available. Figure 13 shows areas of expected overexcavation and deep foundation system with bridge locations. In general, excavations are anticipated to be in either fill or alluvial materials that are present along the majority of the channel systems. Excavations in these types of materials are anticipated to be performed with conventional construction equipment. As discussed above, undocumented well casings from oil and gas extraction could be encountered during construction. Similarly, in fill areas, debris and remnants from demolished structures can be encountered.

Reuse of the excavated material for backfill should be evaluated based on the actual locations of the proposed excavation sites. Note that peat and other organic materials are not suitable for use as backfill. In general, non-organic, granular alluvial materials should be acceptable for reuse, provided that oversize and organic materials are removed and the backfill is appropriately moisture-conditioned and compacted to at least 90% of maximum dry density or greater, as required by final design and considering anticipated use of the site. As general guidance, material with a liquid limit less than 40 and a plastic limit less than 12, or alternatively, with a sand equivalent less than 30, would likely be acceptable. Generally, this excludes clays with moderate to high plasticity, but may allow the reuse of some low plasticity clays and silts. Actual requirements would depend on the soil properties and design criteria. Project-specific and/or site-specific earthwork specifications should be developed prior to construction. All earthwork must meet local building code requirements.

Planning temporary excavations to facilitate the construction of proposed upgrades along the channel should consider the following:

- Proximity to adjacent structures and availability of open space
- Presence of groundwater at shallow depths of about 5 to 15 feet
- Potential for sloughing and flowing sands
- The need for dewatering the bottom of the excavation
- The need for underpinning of existing structures
- Type of temporary shoring

Temporary excavations to a depth of 4 feet can generally be cut vertically, with the exception of locations with relatively loose or running sands. All excavations deeper than 4 feet should be shored or sloped back for safety. As a general recommendation, if space is available and the excavations are at a distance of more than twice the width of the proposed channel modifications from adjacent features and above the groundwater level, excavations can be made with slopes of 2:1 (horizontal to vertical) to a maximum depth of 15 feet. The excavations are generally anticipated to be in loose to medium dense sand and silty sand soils or soft to medium stiff silty clay and silty soil. The 2:1 slopes can be used, provided that the temporary slope excavations are monitored during construction and additional support measures are available in the event of excessive sloughing. It is important that all surface water be directed away from excavation slopes so as to reduce the chance of erosion and seepage. All excavations should be completed in accordance with applicable regulations, including Cal/OSHA guidelines. Note that local subsurface conditions may require flatter slopes, and these recommendations should only be used for planning purposes.

Temporary shoring will likely be necessary for most excavation locations because the existing channels are located in a densely populated area, with most reaches developed adjacent to existing structures. Furthermore, groundwater is shallow throughout the project area (commonly less than 15 feet bgs), and loose to medium dense sand and silty sand is also prevalent in the near-surface stratigraphy of most of the boring logs and CPT signatures we reviewed. Therefore, flowing sands are also a potential issue during excavation and open-cut trenching may not be feasible. In these areas, which constitute most of the project, temporary shoring such as speed-shores, trench boxes, cantilever sheet piles, soldier piles with lagging and tie-backs, and internal bracing could be used throughout the alignment. Non-interlocking shoring would likely not be appropriate in areas with shallow groundwater and sandy materials that would tend to produce running ground conditions.

Cantilevered shoring should be designed for active lateral earth pressures, taking into consideration horizontal or inclined ground surfaces behind the walls.

Braced or tied-back shoring is recommended to support the sides of proposed excavations deeper than 15 feet. For the design of braced or tied-back shoring, it is recommended that lateral earth pressure distribution be used accommodating level or sloped backfill conditions, as appropriate. Soldier pile shoring consisting of steel beams placed in drilled holes, backfilled with concrete, and braced or restrained by tie-back anchors can be used to support the excavations. Lagging will be required between the soldier piles. Soldier piles should be installed at a minimum spacing of three diameters (center to center). For the design of soldier piles spaced at least three diameters on-center, allowable lateral bearing (passive pressure) below the bottom of the proposed excavation should be developed for piles embedded in native soils. Due to the granular nature of the subsurface materials, continuous lagging should be used.

In addition to the lateral earth pressures described above, the upper 10 feet of the shoring adjacent to traffic areas should be designed to resist a uniform lateral surcharge pressure behind the shoring due to street traffic (if applicable). In addition, any surcharge (live or dead load) located within a 1:1 (horizontal to vertical) plane drawn upward from the base of the shored excavation should be added to the lateral earth pressures.

Lateral loads can be resisted by tie-back friction anchors. For the design of pressure-grouted anchors, grout-to-ground bond strength along the bonded zone should be developed. The capacities of the anchors should be verified by testing after installation.

It should be noted that the use of tie-back anchors for the soldier piles should take into consideration existing property lines of neighboring developments and the space available to install the anchors along any permissions from neighboring properties. The installation of the anchors and the testing of the completed anchors should be performed following accepted requirements and procedures and should be observed by a representative of a qualified geotechnical firm.

Predicting actual deflections of a shored embankment is difficult given the complex nature of the construction environment. It should, however, be realized that some deflection is likely to occur and should be estimated, monitored and documented.

8.9 DEWATERING

Groundwater depths in most reaches generally vary between 5 and 15 feet below existing grade. It is also noted that the historically highest groundwater levels were reported at depths ranging from 3 to 30 feet below the existing ground surface. Depending when construction will occur, groundwater levels may vary significantly from those measured in the past during various investigations and may rise to shallower depths.

For areas where the groundwater is below or near the bottom of the proposed excavations, control of groundwater can likely be accomplished by pumping from sumps within the excavation. However, as discussed previously, shallow groundwater levels were reported throughout the project area; as such dewatering is likely required for most, if not all, reaches of the channel systems. Dewatering for such shallow ground water cases will likely require the design and installation of well point systems capable of lowering the groundwater to allow the construction of the proposed upgrades. It is generally recommended that the water levels be lowered to 5 feet below the bottom of the excavation.

In areas where the groundwater will be near or above the excavation bottom, the bottom of the excavations are anticipated to be relatively soft and wet. Consequently, remedial measures will be required to stabilize the bottom to support various construction work and equipment traffic. Such measures can include the placement of geofabric at the bottom of the excavation along with a layer of ¾-inch crushed rock, or alternatively, using geogrid with rock. Depending on the type of construction equipment used and actual conditions of exposed excavation bottom, the thickness of the crushed rock may vary.

It should be noted that the design of the dewatering system should assess the influence zone and the potential impact on adjacent structures due to settlement. Based on the data available, loose or soft soils are also common throughout the project area that are susceptible to settlement and surface deformation due to dewatering. As documented in Report No. 17 in Table 2, sinkholes reportedly formed near Slater Pump Station in early 2011 due to dewatering. Prior to construction, a groundwater monitoring program should be established to monitor drawdown and surface settlements. Localized subsidence or sinkholes related to settlement of shallow, loose or soft, saturated soils during dewatering or structural loading is possible and has been observed in the project area (e.g., at Slater Pump Station; see Report No. 17 in Table 2).

It should also be noted that controlling and maintaining of the groundwater during construction is an important part of dewatering operations. As part of the groundwater control, the quality of collected water should also be checked in accordance with applicable permits and regulations for water discharge.

8.10 FUTURE FIELD EXPLORATION AND LABORATORY TESTING

As shown on Figure 6a, geotechnical information is available for most reaches of the channel systems. However, the information varies in both quality and vintage. In general, recent site-specific studies completed for channel improvements in the coastal reaches of C05 are relatively complete and consistent with the current geotechnical state of practice as opposed to older and incomplete datasets for the eastern reaches. Based on this and in view of the key geotechnical conditions identified, it is recommended that a site specific geotechnical investigation be performed for all reaches of proposed channel improvement to either update the existing information or fill-in the gaps in data and most importantly assist in the design and the construction of the proposed upgrades. While a detailed scope of field exploration cannot be prepared at this time, preliminary general recommendation for future geotechnical investigations are provided for USACE's consideration:

- Once the upgrade plan of the channel is established, a review of the available reports should be performed to plan the project-specific geotechnical investigation. Pertinent existing investigation points can be selected to supplement the project-specific investigation. Depending on the proposed upgrades to the flood channels, field explorations should be planned at spacings between 500 feet and 1,000 feet along the channel. At major facility locations, such as the proposed basin at Mile Square Park or the location of bridges and large culverts, the number of the investigation points should focus on configuration of the structure, size, and loading conditions.

- The field explorations can generally be accomplished using borings, CPTs, and test pits. Depending on the anticipated subsurface conditions, either hollow-stem auger or rotary wash borings can be considered, along with CPTs. Test pits can be used at channel bottoms or accessible locations. The depth of the borings and CPTs will depend on the type of proposed channel and configuration; in general, the depth of the investigation points should extend to a depth equal to at least one times the width of the proposed upgraded channel width. Note that local conditions could require deeper borings. The borings should be logged, and groundwater should be measured and monitored by installation of piezometers. The wells should be developed by surging, bailing, and/or pumping.
- Samples from the borings should be obtained using SPT samplers, Modified California samplers, and Shelby tubes at approximately 5-foot intervals within hollow-stem auger or rotary wash boreholes.
- Geotechnical laboratory testing on select retrieved samples should be performed for soil characterization and to evaluate static physical soil properties. Tests may include moisture content, sieve analysis, wash analysis, Atterberg limits, expansion index, unconfined compression, direct shear, triaxial, consolidation, and corrosion. If critical structures are in areas susceptible to liquefaction or cyclic softening, additional testing could be required.

8.11 MONITORING DURING CONSTRUCTION

Due to the proximity of existing buildings and structures, construction should be performed carefully. An initial survey should be taken prior to the excavation so that an accurate baseline may be established. We recommend that the initial survey and monitoring program also include the adjacent existing structures. Photographs and videos of the existing permanent structures are recommended as part of the documentation process.

It is also recommended that some means of monitoring the performance of the shoring system be performed. The monitoring should at a minimum consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier or sheet piles.

9.0 LIMITATIONS

The observations, conclusions, and recommendations presented in this appendix are based upon our review of the documents provided to us and our relevant previous experience. No field exploration, laboratory testing, or engineering analyses were performed as part of this review. In addition, we have relied on data such as boring logs or groundwater levels reported by others. As such, the findings summarized in this appendix are preliminary and subject to change when additional information or further investigations become available.

The information presented in this report is intended to be used for project planning purposes only. This information is subject to change once the specific details or features of the proposed project are identified, updated, and/or modified.

Professional judgments presented in this report are based on an evaluation of the technical information gathered and GeoPentech's general experience in the fields of geotechnical engineering and geology. GeoPentech does not guarantee the performance of the project in any respect, only that the engineering work and judgment rendered herein meet the standard of care of the geotechnical profession at this time.

10.0 REFERENCES

(See Table 2 for citations of reports and plans reviewed for this preliminary geotechnical appendix. Additional citations are listed below.)

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TABLES

TABLE 1
USACE PROPOSED ALTERNATIVES
WESTMINSTER CHANNELS PROJECT

Alternative	Description
2	Convert various reaches from existing trapezoidal channels with natural inverts and natural or rip-rap sidewalls to concrete trapezoidal channels.
3	Add three-foot-high floodwalls to some existing reaches. Convert some reaches from existing trapezoidal channels with natural inverts and concrete or rip-rap sidewalls to concrete rectangular channels (with or without three-foot-high floodwalls, varies by location).
4	Convert most reaches to concrete rectangular channels (with or without three-foot-high floodwalls, varies by location). Convert some reaches from existing trapezoidal channels with natural inverts and natural or rip-rap sidewalls to concrete trapezoidal channels or rectangular channels with natural inverts and sheet pile walls.
5	Convert various reaches from existing trapezoidal channels with natural inverts and natural or rip-rap sidewalls to concrete trapezoidal channels. Construct new detention basin in Mile Square Park along C06 channel and new buried conduit under Ward Street, connecting C05 channel to new basin.

TABLE 2
REPORTS AND DOCUMENTS REVIEWED FOR
WESTMINSTER CHANNELS PROJECT

Report No.	Title	Author	Date
1	Final Geotechnical Investigation Report, East Garden Grove-Wintersburg Channel Improvements from Graham Street to Warner Avenue (Station 75+00 to 100+00), Huntington Beach, Orange County, California, 349 pp.	Earth Mechanics, Inc.	10/15/2009
2	Final Geotechnical Investigation Report, East Garden Grove-Wintersburg Channel Improvements from Warner Avenue to Upstream of Edwards Street (Station 101+00 to 151+25), Huntington Beach, Orange County, California, 717 pp.	Earth Mechanics, Inc.	10/15/2009
3	Westminster Feasibility Study Preliminary Draft Baseline Conditions Report, Appendix F, Geotechnical, 21 pp.	U.S. Army Corps of Engineers	5/1/2007
4	Project Report, Bolsa Chica Channel from Huntington Harbor Outlet to Cerritos Avenue, 270 pp.	Robert Bein, William Frost & Associates	6/24/1983
5	Geotechnical Evaluation, Westminster Channel Improvements, Huntington Beach, California, 189 pp.	Ninyo & Moore	5/24/2011
6	Final Geotechnical Investigation Report, Westminster Channel Improvements (C04) from Hoover Street to Beach Boulevard, County of Orange, California, 68 pp.	Earth Mechanics, Inc.	1/11/2007
7	Plans for Construction of East Garden Grove - Wintersburg Channel North Levee Emergency Project from 3800 feet Downstream of Graham St. to Graham St., 22 pp.	Orange County Resources and Development Management Dept.	10/15/2007
8	East Garden Grove - Wintersburg Channel, OCFCD Facility C05, Slope Stability Analysis of the North Levee Downstream of the Oil Bridge, 43 pp.	Orange County Flood Control District	9/23/2008
9	Geotechnical Investigation Report for East Garden Grove Wintersburg Channel (OCFCD Facility C05) Improvements Phase 1, Proposed Sheet Pile Buttress Support from STA. 34+00 to STA. 53-16, City of Huntington Beach, Orange County, California, 163 pp.	URS Corpoation	1/18/2011
10	Geotechnical Investigation, East Garden Grove - Wintersburg Channel (C05) Levee Soil Mix Project, Groundwater Impact Evaluation, Station 37+00 to Station 102+00, Huntington Beach, California, 283 pp.	Hushmand Associates, Inc.	5/20/2010
11	Final Geotechnical Investigation Report, East Garden Grove - Wintersburg Channel Improvements from Graham Street to Warner Avenue (Station 75+00 to 100+00), Huntington Beach, Orange County, California, 343 pp.	Earth Mechanics, Inc.	10/15/2009
12	Deep Soil Mix Column Levee Structure, East Garden Grove - Wintersburg Channel Improvement, Huntington Ceach, Orange County, California, 29 pp.	Earth Mechanics, Inc.	5/5/2008
13	Geotechnical Engineering Investigation, East Garden Grove - Wintersburg Channel (C05) Emergency Project, North Levee, Station 36+00 to Station 50+00, Huntington Beach, Orange County, California, 170 pp.	Hushmand Associates, Inc.	12/28/2007
14	Geotechnical Review and Feasibility Evaluation, Proposed Levee Improvements, East Garden Grove - Wintersburg Channel Station 48+00 to 74+25 (C05), 92 pp.	Advanced Earth Science, Inc.	6/29/2005
15	Final Report of Geotechnical Investigation, Proposed East Garden Grove Wintersburg Channel C05 Improvement, Southwest of Graham Street, Southern Levee, from Station No. 48+00 to Station No. 74+25, Huntington Beach, California, 275 pp.	MACTEC Engineering and Consilting, Inc.	12/2/2004
31	Plans for Construction of East Garden Grove - Wintersburg Channel from Bolsa Chica Tide Gates to Upstream of Warner Ave. (from Station 6+34 to Station 102+02), 62 pp.	Orange County Public Works Dept.	7/1/2010

Report No.	Title	Author	Date
16	Geotechnical Investigation Materials Report, East Garden Grove Wintersburg Channel C05 Station 50+00 to Station 152+00, 131 pp.	Orange County Public Facilities & Resources Dept.	3/1/2001
17	Review of Water Quality Enhancementa nd Perched Water Buffer Components for East Garden Grove - Wintersburg Channel (C05) Levee Improvements, 9 pp.	WRC Consulting Services, Inc.	5/23/2012
18	East Garden Grove - Wintersburg Channel OCFCD Facility #C05, Quantitative Engineering Analysis of North Levee Downstream of Graham Street, 31 pp.	Orange County Resources and Development Management Dept.	9/25/2007
19	Report of Geotechnical Investigation, Proposed East Garden Grove Wintersburg Channel C05 Improvement Southwest of Graham Street, Northern Levee From Station No. 48+00 to Station No. 74+25, Huntington Beach, California, Appendices B and C, 44 pp.	MACTEC Engineering and Consilting, Inc.	12/2/2004
20	Report of Geotechnical Investigation, Proposed East Garden Grove Wintersburg Channel C05 Improvement Southwest of Graham Street, Northern Levee From Station No. 48+00 to Station No. 74+25, Huntington Beach, California, Main Report and Appendix A, 55 pp.	MACTEC Engineering and Consilting, Inc.	12/2/2004
21	Excerpt from Unknown Source, Map of Westminster Feasibility Phase Study, North/Western Orange County, CA, 1 pp.	unknown	unknown
22	Excerpt from Unknown Source, Map of Westminster Feasibility Phase Study, North/Western Orange County, CA, 1 pp.	unknown	unknown
23	Excerpt from Unknown Source, Map of Westminster Feasibility Phase Study, North/Western Orange County, CA, 1 pp.	unknown	unknown
24	Excerpt from Unknown Source, Map of Westminster Feasibility Phase Study, North/Western Orange County, CA, 1 pp.	unknown	unknown
25	Excerpt from Unknown Source, Map of Westminster Feasibility Phase Study, North/Western Orange County, CA, 1 pp.	unknown	unknown
26	Excerpt from Unknown Source, Map of Westminster Feasibility Phase Study, North/Western Orange County, CA, 1 pp.	unknown	unknown
27	Excerpt from "Final Report of Geotechnical Investigation, Proposed East Garden Grove Wintersburg Channel C05 Improvement, Southwest of Graham Street, Southern Levee, from Station No. 48+00 to Station No. 74+25, 39 pp.	MACTEC Engineering and Consilting, Inc.	12/4/2004
28	Preliminary Geotechnical Investigation, Proposed Residential Development, Tentative Tract 15377, City of Huntington Beach, California, and Tentative Tract 15419, County of Orange, California, 264 pp.	Pacific Soils Engineering, Inc.	2/2/1998
29	Westminster Feasibility Study Preliminary Draft Baseline Conditions Report, 161 pp.	U.S. Army Corps of Engineers	5/1/2007
30	Report Synopsis for Westminster, East Garden Grove Flood Risk Management Study, 40 pp.	unknown	2/1/2014
46	Plans for the Construction of That Portion of Ocean View Channel, Facility C06, Magnolia Street to Bushard Street, 10 pp.	Orange County Environmental Management Agency	5/1/1983

TABLE 2
REPORTS AND DOCUMENTS REVIEWED FOR
WESTMINSTER CHANNELS PROJECT

Report No.	Title	Author	Date
32	Project Report for Westminster Channel (C04), Bolsa Chica Confluence (C02) to Garden Grove Freeway (SR-22), 237 pp.	WRC Consulting Services, Inc.	2/1/2005
33	Project Report for East Garden Grove - Wintersburg (C05) and Oceanview (C06) Channels, 279 pp.	Williamson & Schmid	12/1/1994
34	As-Built Plans for the Construction of Bolsa Chica Channel, Tidelands to Cerritos Avenue, 32 pp.	Orange County Flood Control District	6/1/1959
35	As-Built Plans for Construction of Bolsa Chica Channel (Facility No. C02) from Westminster Channel (Sta. 86+00) to Anaheim-Barber City Channel (Sta. 166+00), 8 pp.	Orange County Environmental Management Agency	1/1/1985
36	Plans for the Construction of Westminster Channel, McFadden Avenue to Sta. 92+20 and at Graham Street, 10 pp.	Orange County Flood Control District	12/1/1962
37	Plans for Construction of Westminster Channel, Facility No. C04, from D/S Goldenwest Street to U/S San Diego Fwy. (I-405), 11 pp.	Orange County Environmental Management Agency	2/1/1991
38	Plans for Construction of Westminster Channel, Facility No. C04, from U/S of Magnolia to D/S of Brookhurst, 12 pp.	Orange County Environmental Management Agency	3/1/1992
39	As-Built Plans for the Construction of East Garden Grove–Wintersburg Channel, Tidelands to Huntington Beach Blvd., 30 pp.	Orange County Flood Control District	9/1/1959
40	Plans for the Construction of East Garden Grove–Wintersburg Channel, Beach Blvd. to Newhope Street, 25 pp.	Orange County Flood Control District	4/1/1960
41	Plans for the Construction of East Garden Grove-Wintersburg Channel, Newhope St. to Haster Retarding Basin, 22 pp.	Orange County Flood Control District	6/1/1961
42	Plans for the Southern Levee Restoration of East Garden Grove Wintersburg Channel, Facility No. C05, from Graham St. to Warner Ave., 13 pp.	Orange County Environmental Management Agency	9/1/1993
43	Plans for the Construction of East Garden Grove–Wintersburg Channel O.C.F.C.D. Facility No. C05, from 411 m D/S of Golden West St. to 349 m U/S of Golden West St. [sic], 33 pp.	Orange County Environmental Management Agency	4/1/1998
44	Plans for the Construction of East Garden Grove–Wintersburg Channel O.C.F.C.D. Facility No. C05, and Ocean View Channel O.C.F.C.D. Facility No. C06, from 349 m D/S of Golden West St. [sic] to 350 m U/S of Gothard St., 37 pp.	Orange County Environmental Management Agency	2/1/2001
45	Plans for Improvement of Newland Storm Channel, C05 Confluence to D/S Whitley Ave (Facility No. C05S01), Station 00+34.11 to Station 61+40, 79 pp.	County of Orange Public Works	1/1/2014
47	Geotechnical Services, Soil Sampling and Laboratory Testing, Huntington Harbour, California, 20 pp.	Diaz · Yourman & Associates	6/21/1994
48	Liquefaction Potential, Sycamore Valley Apartment Complex, 10349 Slater Avenue, Fountain Valley, California, 35 pp.	Diaz · Yourman & Associates	2/8/1996
49	Draft Geotechnical Services for Southern California Edison Fenwick Building Modifications, 14799 Chestnut Street, Westminster, California, 2 pp.	Diaz · Yourman & Associates	3/1/2000
50	Geotechnical Investigation, Chlorine Containment Equipment Shelters, Wells 6, 7, 9, and 10, Huntington Beach, California, 36 pp.	Diaz · Yourman & Associates	9/15/2000
51	Initial Site Assessment, Seal Beach Regional Trail, Seal Beach, California, 188 pp.	Diaz · Yourman & Associates	4/5/2004
52	Geotechnical Investigation, OCTA Annex Building, Garden Grove, California, 72 pp.	Diaz · Yourman & Associates	5/20/2005
53	Geotechnical Investigation, Moran Street and Bishop Place, Westminster, California, 24 pp.	Diaz · Yourman & Associates	6/1/2007
54	Geotechnical Investigation, Petroleum Storage Tank, Huntington Beach, California, 71 pp.	Diaz · Yourman & Associates	8/8/2008
55	Geotechnical Investigation, Street and Drainage Improvements, Westminster, California, 40 pp.	Diaz · Yourman & Associates	6/5/2012
56	Geotechnical Investigation, Bulkhead Evaluation, Sunset Harbor Maintenance Dredging, Orange County, California, 109 pp.	Diaz · Yourman & Associates	9/27/2013
57	Geotechnical Investigation, Gothard-Hoover Street Extension, Orange County, California, 87 pp.	Harding Lawson Associates	7/31/1990
58	Geotechnical Investigation, Holiday Inn Hotel, Center Drive, Huntington Beach, California, 92 pp.	Woodward-Clyde Consultants	3/20/1984
59	Geotechnical Engineering Investigation, Proposed Sam's Club with Fueling Facility and Two Satellite Pads, SWC of Brookhurst Street and Warner Avenue, Fountain Valley, California, 117 pp.	Krazen & Associates, Inc.	3/2/2005

TABLE 3
GEOLOGIC CONDITIONS SUMMARY
WESTMINSTER CHANNELS PROJECT

Channel	Reach	Depth to Historically Highest Groundwater[1]	Mapped Seismic Hazards[2]	Geologic Unit Abbreviation[3]	Geologic Unit Name[3]	Geologic Unit Description[3]	Distribution of Peat[4]	Distribution of Non-Peat Organics[4]	Distribution of Soft Soils[4]
C05	1	Very Shallow (0 to 5 ft bgs)	Liquefaction Zone of Required Investigation Alquist-Priolo Earthquake Fault Zone	af	Artificial Fill	deposits of fill resulting from human construction, mining, or quarrying activities; includes engineered fill for buildings, roads, dams, airport runways, harbor facilities, and waste landfills	Generally Encountered at Depths of 0 ft bgs to Deeper than 25 ft bgs	Generally Encountered at Depths of 0 ft bgs to Deeper than 25 ft bgs	Generally Encountered at Depths of 0 ft bgs to Deeper than 25 ft bgs
				Qb	Beach Deposits	unconsolidated marine beach sediments consisting mostly of fine- and medium-grained, well-sorted sand			
				Ql	Lacustrine, Playa, and Estuarine (Paralic) Deposits	mostly unconsolidated fine-grained sand, silt, mud, and clay from fresh water (lacustrine) lakes, saline (playa) dry lakes that are periodically flooded, and estuaries; deposits may contain salt and other evaporites			
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
				Qyl	Young Lacustrine, Playa, and Estuarine (Paralic) Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarinedeposits of various types			
				Qol	Old Lacustrine, Playa, and Estuarine (Paralic) Deposits	slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types			
C05	2	Very Shallow (3 to 10 ft bgs)	Liquefaction Zone of Required Investigation	Qsu	Undifferentiated Surficial Deposits	includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers	Generally Encountered at Depths of 0 ft bgs to Deeper than 25 ft bgs	Locally Encountered at Depths of 0 ft bgs to Deeper than 25 ft bgs	Generally Encountered at Depths of 0 ft bgs to Deeper than 25 ft bgs
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
				Qyl	Young Lacustrine, Playa, and Estuarine (Paralic) Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarinedeposits of various types			
				Qol	Old Lacustrine, Playa, and Estuarine (Paralic) Deposits	slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types			
C05	3	Very Shallow (5 to 10 ft bgs)	Liquefaction Zone of Required Investigation	Qsu	Undifferentiated Surficial Deposits	includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers	Generally Encountered at Depths of 0 ft bgs to Deeper than 25 ft bgs	Generally Encountered at Depths of 0 ft bgs to Deeper than 25 ft bgs	Generally Encountered at Depths of 0 ft bgs to Deeper than 25 ft bgs
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
				Qyl	Young Lacustrine, Playa, and Estuarine (Paralic) Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarinedeposits of various types			
				Qol	Old Lacustrine, Playa, and Estuarine (Paralic) Deposits	slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types			
C05	4	Very Shallow (0 to 5 ft bgs)	Liquefaction Zone of Required Investigation	Qsu	Undifferentiated Surficial Deposits	includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs	Locally Encountered at Depths of 0 ft bgs to 25 ft bgs	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs; Locally Encountered at Depths Deeper than 25 ft bgs
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
				Qol	Old Lacustrine, Playa, and Estuarine (Paralic) Deposits	slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types			
C05	5	Very Shallow (0 to 5 ft bgs)	Liquefaction Zone of Required Investigation	Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs	Not Reported in Previous Studies	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
C05	6	Very Shallow (0 to 5 ft bgs)	Liquefaction Zone of Required Investigation	Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs	Not Reported in Previous Studies	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
C05	7	Very Shallow (0 to 5 ft bgs)	Liquefaction Zone of Required Investigation	Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs	Not Reported in Previous Studies	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			

TABLE 3
GEOLOGIC CONDITIONS SUMMARY
WESTMINSTER CHANNELS PROJECT

Channel	Reach	Depth to Historically Highest Groundwater[1]	Mapped Seismic Hazards[2]	Geologic Unit Abbreviation[3]	Geologic Unit Name[3]	Geologic Unit Description[3]	Distribution of Peat[4]	Distribution of Non-Peat Organics[4]	Distribution of Soft Soils[4]
C05	8	Very Shallow (0 to 10 ft bgs)	Liquefaction Zone of Required Investigation	Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon	Not Reported in Previous Studies	Not Reported in Previous Studies	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
C05	9	Very Shallow (5 to 20 ft bgs)	Liquefaction Zone of Required Investigation	Qw	Alluvial Wash Deposits	unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand	Not Reported in Previous Studies	Not Reported in Previous Studies	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
C05	10	Shallow (10 to 30 ft bgs)	Liquefaction Zone of Required Investigation	Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon	Not Reported in Previous Studies	Not Reported in Previous Studies	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs
C06	13	Very Shallow (5 to 10 ft bgs)	Liquefaction Zone of Required Investigation	Qsu	Undifferentiated Surficial Deposits	includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs	Not Reported in Previous Studies; No Site-Specific Data Available in Some Areas	Generally Encountered at Depths of 0 ft bgs to Deeper than 25 ft bgs; No Site-Specific Data Available in Some Areas
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
				Qyl	Young Lacustrine, Playa, and Estuarine (Paralic) Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarinedeposits of various types			
				Qol	Old Lacustrine, Playa, and Estuarine (Paralic) Deposits	slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types			
C06	14	Very Shallow (5 to 10 ft bgs)	Liquefaction Zone of Required Investigation	Qsu	Undifferentiated Surficial Deposits	includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers	No Site-Specific Data Available	No Site-Specific Data Available	No Site-Specific Data Available
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
				Qol	Old Lacustrine, Playa, and Estuarine (Paralic) Deposits	slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types			
C06	15	Very Shallow (0 to 5 ft bgs)	Liquefaction Zone of Required Investigation	Qsu	Undifferentiated Surficial Deposits	includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers	No Site-Specific Data Available	No Site-Specific Data Available	No Site-Specific Data Available
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
				Qol	Old Lacustrine, Playa, and Estuarine (Paralic) Deposits	slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types			
C06	16	Very Shallow (0 to 5 ft bgs)	Liquefaction Zone of Required Investigation	Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs	Not Reported in Previous Studies	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
				Qol	Old Lacustrine, Playa, and Estuarine (Paralic) Deposits	slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types			
C06	17	Very Shallow (0 to 5 ft bgs)	Liquefaction Zone of Required Investigation	Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon	Not Reported in Previous Studies	Not Reported in Previous Studies	Generally Encountered at Depths of 0 ft bgs to 25 ft bgs
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
C06	18	Very Shallow (0 to 5 ft bgs)	Liquefaction Zone of Required Investigation	Qw	Alluvial Wash Deposits	unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand	No Site-Specific Data Available	No Site-Specific Data Available	No Site-Specific Data Available
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			

TABLE 3
GEOLOGIC CONDITIONS SUMMARY
WESTMINSTER CHANNELS PROJECT

Channel	Reach	Depth to Historically Highest Groundwater[1]	Mapped Seismic Hazards[2]	Geologic Unit Abbreviation[3]	Geologic Unit Name[3]	Geologic Unit Description[3]	Distribution of Peat[4]	Distribution of Non-Peat Organics[4]	Distribution of Soft Soils[4]
C06	19	Very Shallow (0 to 5 ft bgs)	Liquefaction Zone of Required Investigation	Qw	Alluvial Wash Deposits	unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand	No Site-Specific Data Available	No Site-Specific Data Available	No Site-Specific Data Available
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
C02 / C04	20	Very Shallow (0 to 5 ft bgs)	Liquefaction Zone of Required Investigation Alquist-Priolo Earthquake Fault Zone	af	Artificial Fill	deposits of fill resulting from human construction, mining, or quarrying activities; includes engineered fill for buildings, roads, dams, airport runways, harbor facilities, and waste landfills	Not Reported in Previous Studies; No Site-Specific Data Available in Some Areas	Not Reported in Previous Studies; No Site-Specific Data Available in Some Areas	Locally Encountered at Depths of 0 ft bgs to 25 ft bgs; No Site-Specific Data Available in Some Areas
				Qsu	Undifferentiated Surficial Deposits	includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers			
				Qb	Beach Deposits	unconsolidated marine beach sediments consisting mostly of fine- and medium-grained, well-sorted sand			
				Ql	Lacustrine, Playa, and Estuarine (Paralic) Deposits	mostly unconsolidated fine-grained sand, silt, mud, and clay from fresh water (lacustrine) lakes, saline (playa) dry lakes that are periodically flooded, and estuaries; deposits may contain salt and other evaporites			
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
				Qol	Old Lacustrine, Playa, and Estuarine (Paralic) Deposits	slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types			
				Qyl	Young Lacustrine, Playa, and Estuarine (Paralic) Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarinedeposits of various types			
				Qvof	Very Old Alluvial Fan Deposits	moderately to well-consolidated, highly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
C04	21	Very Shallow (0 to 5 ft bgs)	Liquefaction Zone of Required Investigation	Qsu	Undifferentiated Surficial Deposits	includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers	Locally Encountered at Depths of 0 ft bgs to Deeper than 25 ft bgs; No Site-Specific Data Available in Some Areas	Not Reported in Previous Studies; No Site-Specific Data Available in Some Areas	Locally Encountered at Depths of 0 ft bgs to 25 ft bgs; No Site-Specific Data Available in Some Areas
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
				Qyl	Young Lacustrine, Playa, and Estuarine (Paralic) Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarinedeposits of various types			
				Qvof	Very Old Alluvial Fan Deposits	moderately to well-consolidated, highly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
C04	22	Very Shallow (5 to 20 ft bgs)	Liquefaction Zone of Required Investigation	Qsu	Undifferentiated Surficial Deposits	includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers	No Site-Specific Data Available	No Site-Specific Data Available	No Site-Specific Data Available
				Qyf	Young Alluvial Fan Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			
				Qya	Young Alluvial Valley Deposits	unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers			
				Qvof	Very Old Alluvial Fan Deposits	moderately to well-consolidated, highly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon			

Notes: (1) Based on historically highest groundwater depth contours reported in California Geological Survey (formerly California Division of Mines and Geology) Seismic Hazard Zone Reports.
(2) Based State Regulatory Maps, including Liquefaction Zones of Required Investigation and Alquist-Priolo Earthquake Fault Zones.
(3) Based on California Geological Survey Map of Quaternary Surficial Deposits in Southern California.
(4) Based on review of available reports.

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FIGURES

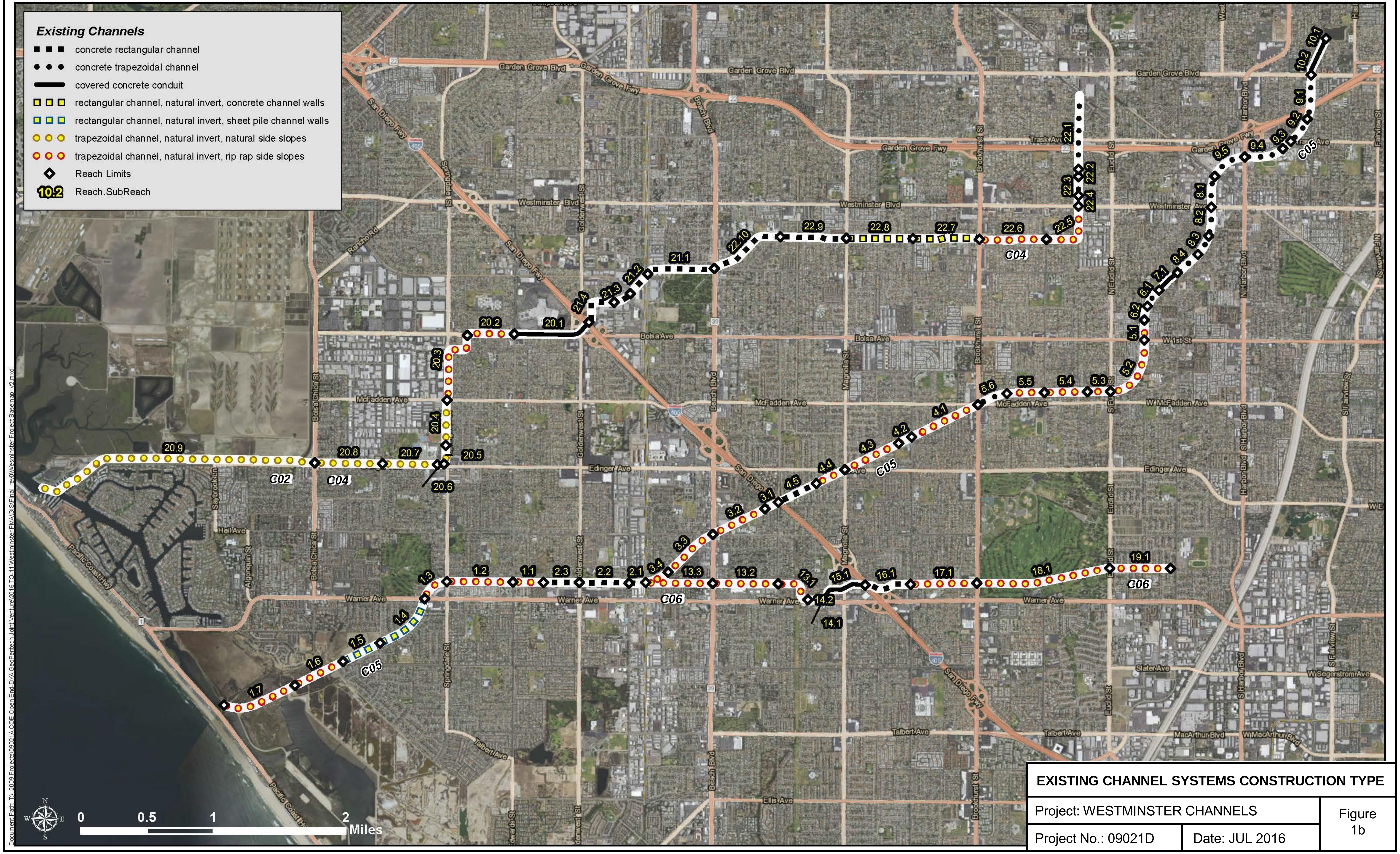
EXISTING CHANNEL SYSTEMS

Project: WESTMINSTER CHANNELS

Project No.: 09021D

Date: JUL 2016

Figure 1a



EXISTING CHANNEL SYSTEMS CONSTRUCTION TYPE

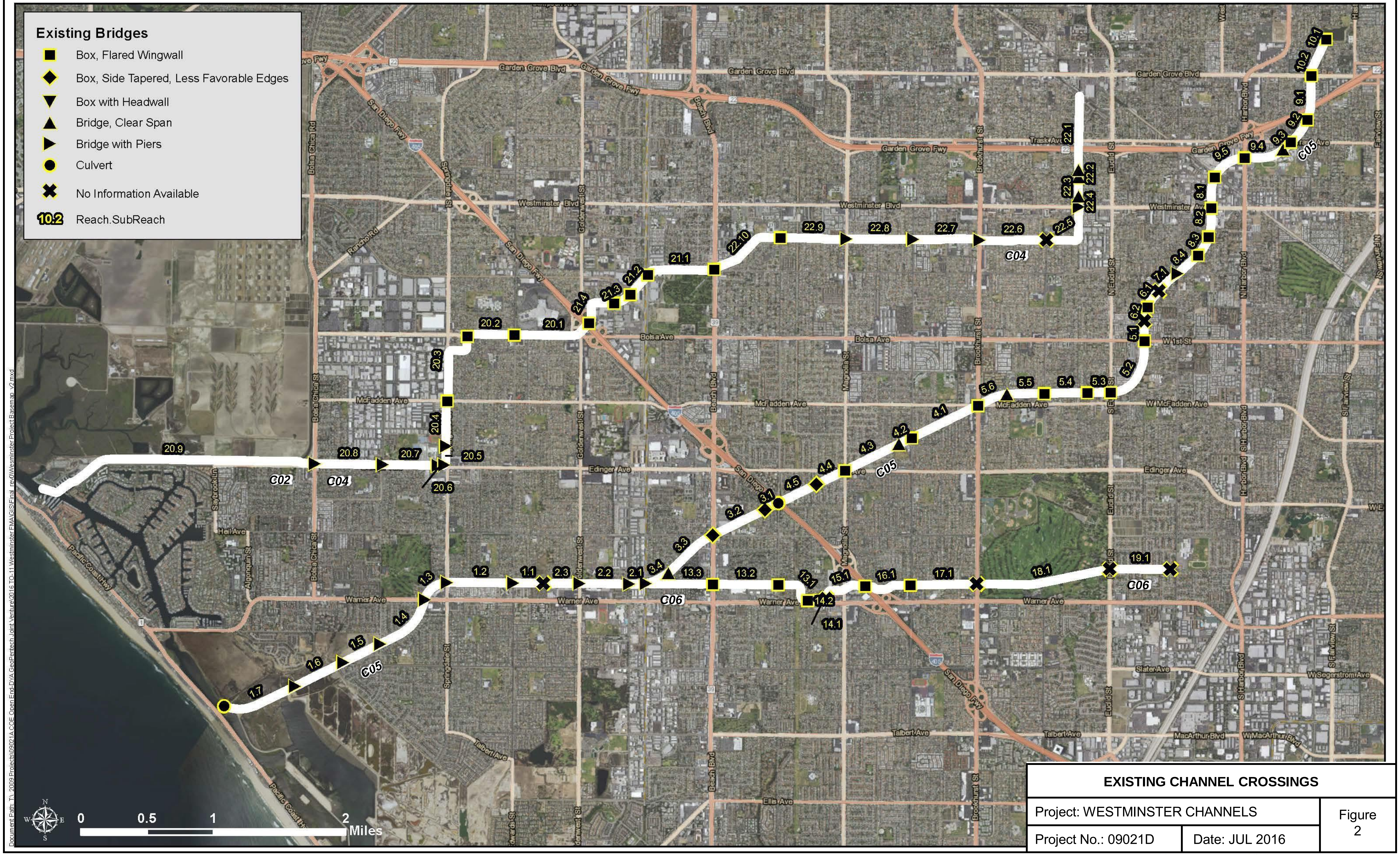
Project: WESTMINSTER CHANNELS

Project No.: 09021D

Date: JUL 2016

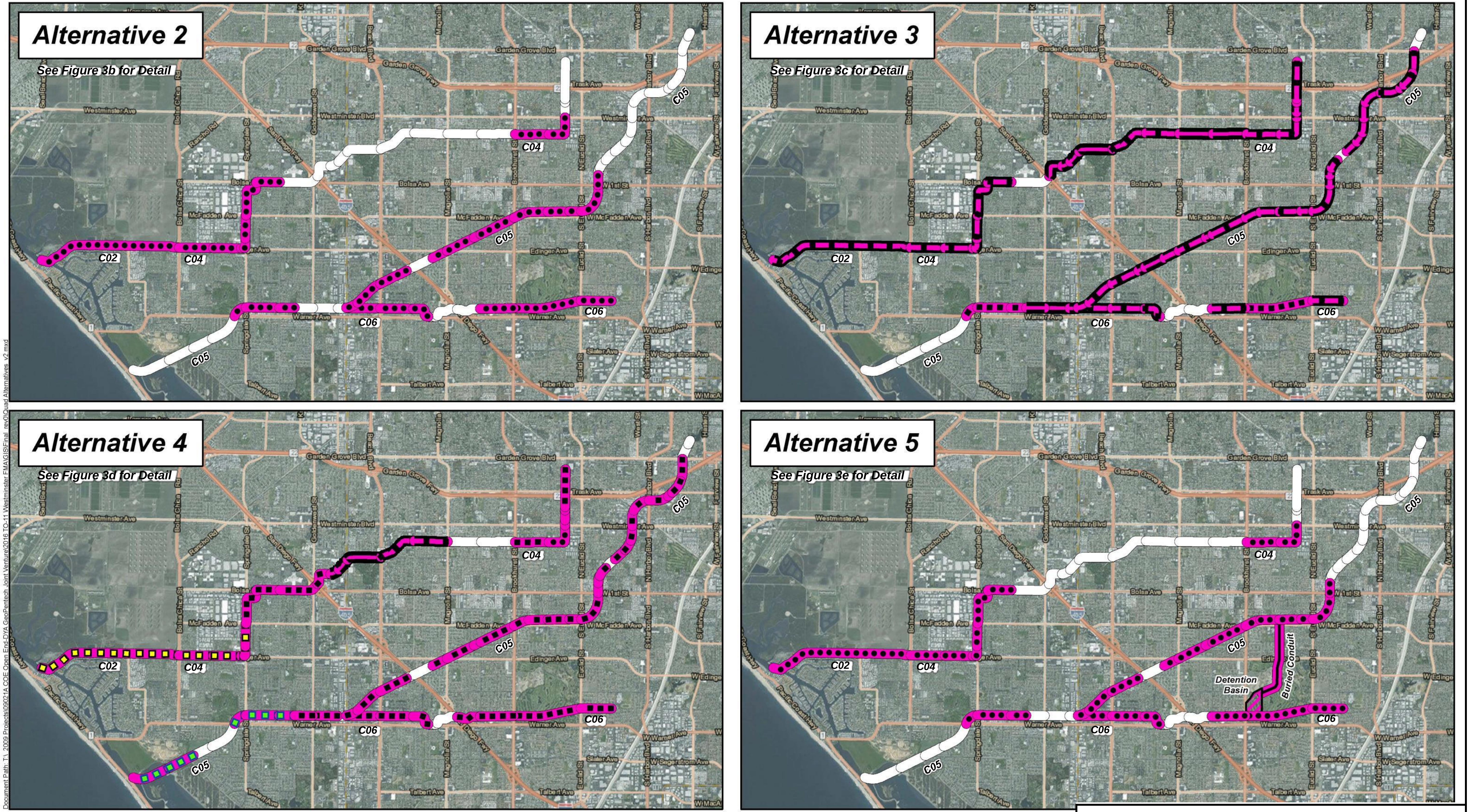
Figure 1b

Document Path: T:_2009 Projects\09021A COE Open End-DYA GeoPentech Joint Venture\2016 TO-11 Westminster FMA\GIS\Final_rev\Westminster Project Basemap_v2.mxd



EXISTING CHANNEL CROSSINGS

Project: WESTMINSTER CHANNELS		Figure 2
Project No.: 09021D	Date: JUL 2016	



Unmodified Reaches

construct concrete rectangular channel, natural invert, concrete channel walls

construct concrete rectangular channel, natural invert, sheet pile channel walls

construct concrete trapezoidal channel

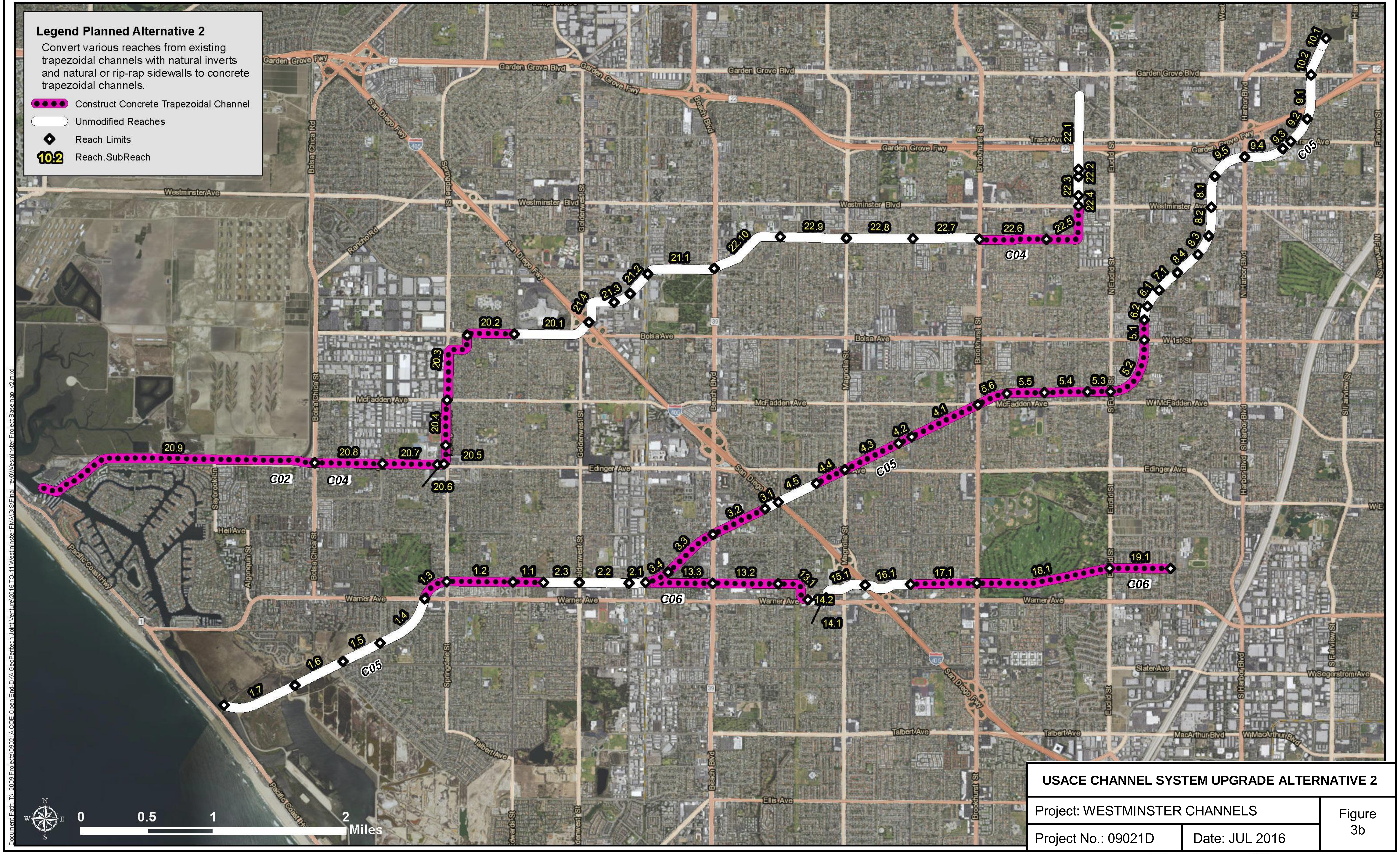
construct concrete rectangular channel

construct 3' floodwall along top of channel

construct concrete rectangular channel; construct 3' floodwall along top of channel

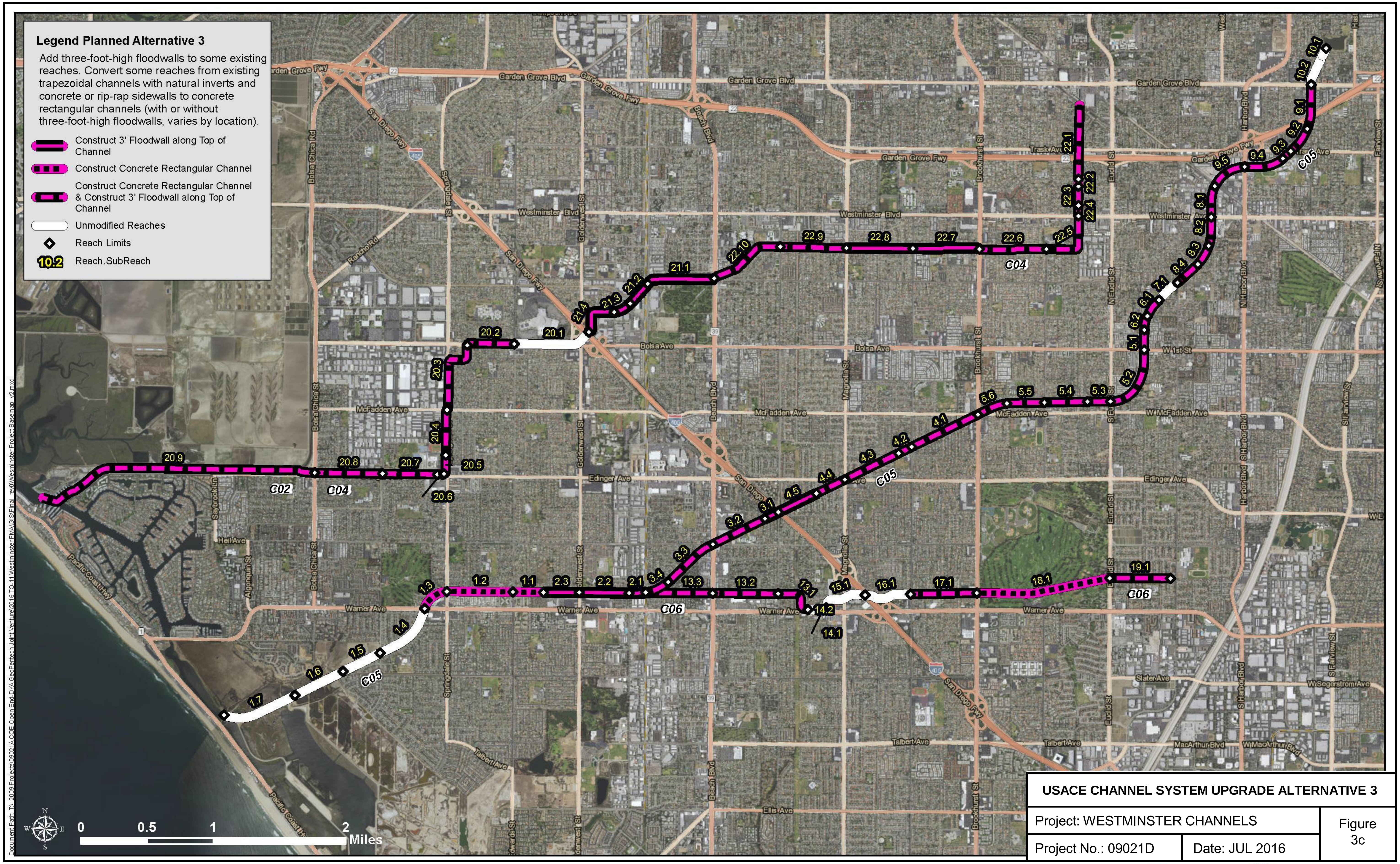
0 0.5 1 2 3 Miles

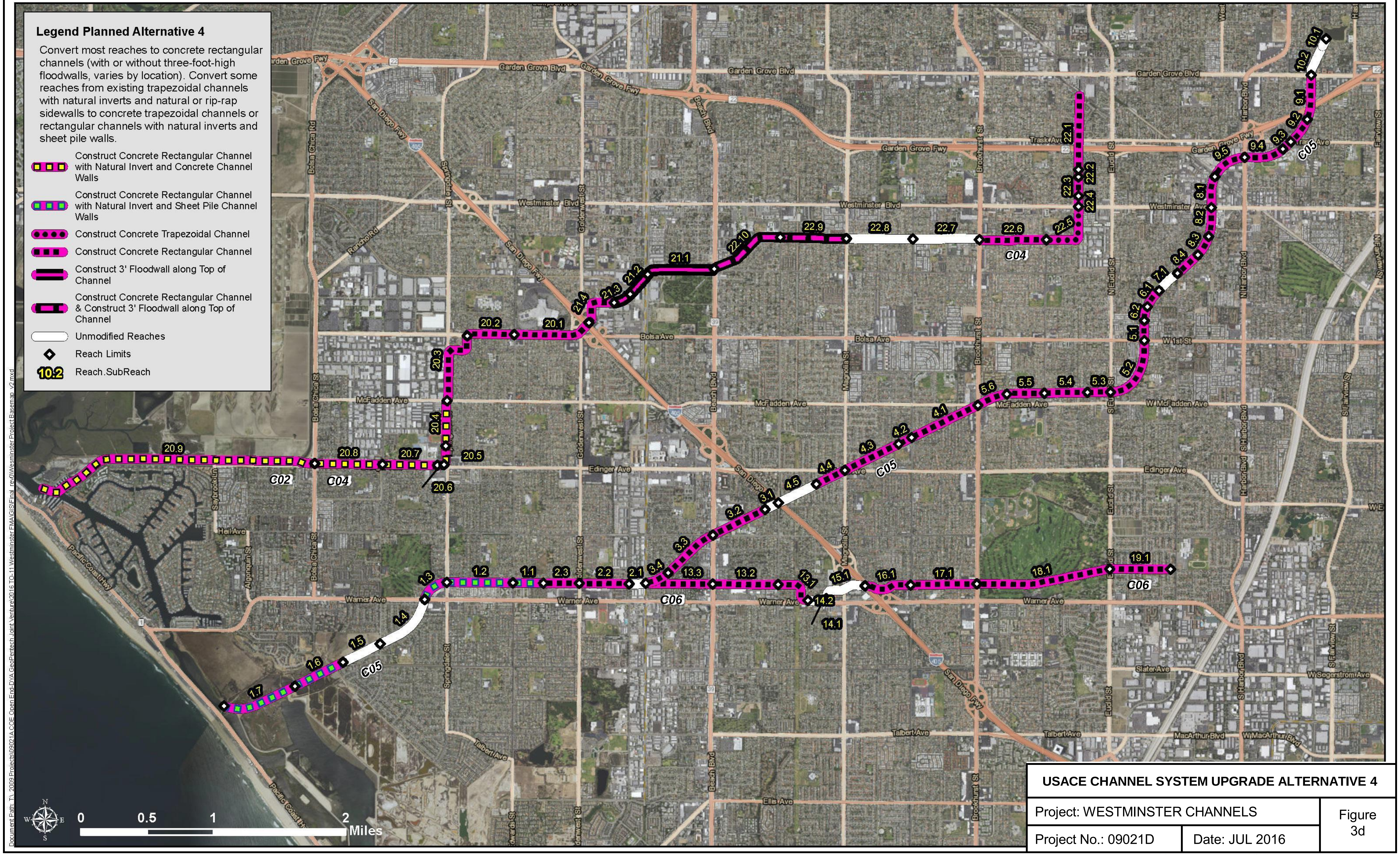
USACE CHANNEL SYSTEM UPGRADE ALTERNATIVES		
Project: WESTMINSTER CHANNELS		Figure 3a
Project No.: 09021D	Date: JUL 2016	



USACE CHANNEL SYSTEM UPGRADE ALTERNATIVE 2

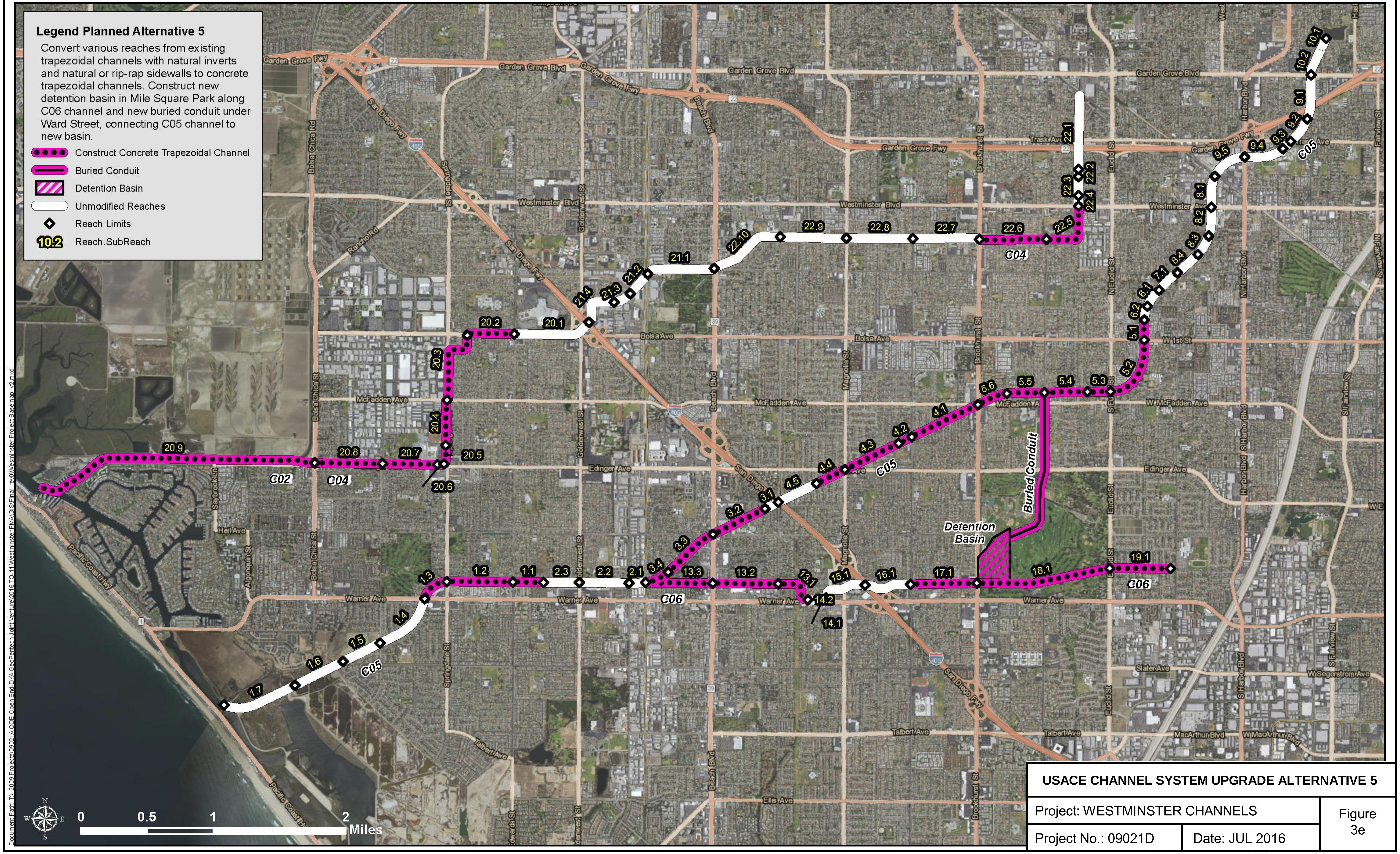
Project: WESTMINSTER CHANNELS	Figure 3b
Project No.: 09021D	Date: JUL 2016


Legend Planned Alternative 3
Add three-foot-high floodwalls to some existing reaches. Convert some reaches from existing trapezoidal channels with natural inverts and concrete or rip-rap sidewalls to concrete rectangular channels (with or without three-foot-high floodwalls, varies by location).
Construct 3' Floodwall along Top of Channel
Construct Concrete Rectangular Channel
Construct Concrete Rectangular Channel & Construct 3' Floodwall along Top of Channel
Unmodified Reaches
Reach Limits
10.2 Reach.SubReach
C02
C04
C05
C06
Garden Grove Blvd
Garden Grove Fwy
Westminster Blvd
Bolsa Ave
McFadden Ave
Edinger Ave
Warner Ave
Talbert Ave
Slater Ave
Ellis Ave
San Diego Fwy
Pacific Coast Hwy
0 0.5 1 2 Miles
USACE CHANNEL SYSTEM UPGRADE ALTERNATIVE 3
Project: WESTMINSTER CHANNELS
Project No.: 09021D
Date: JUL 2016
Figure 3c



USACE CHANNEL SYSTEM UPGRADE ALTERNATIVE 4

Project: WESTMINSTER CHANNELS		Figure 3d
Project No.: 09021D	Date: JUL 2016	



USACE CHANNEL SYSTEM UPGRADE ALTERNATIVE 5

Project: WESTMINSTER CHANNELS		Figure 3e
Project No.: 09021D	Date: JUL 2016	

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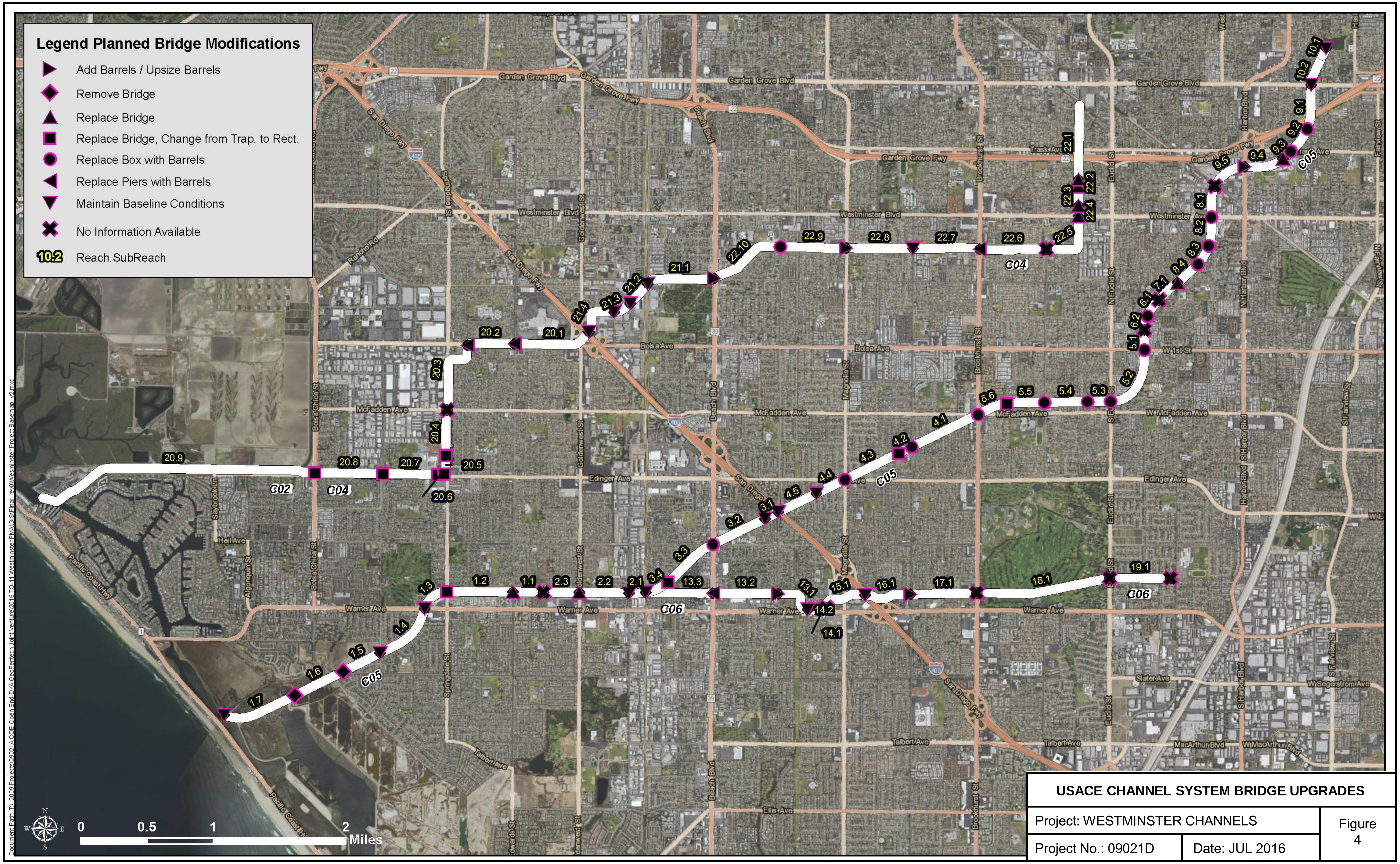
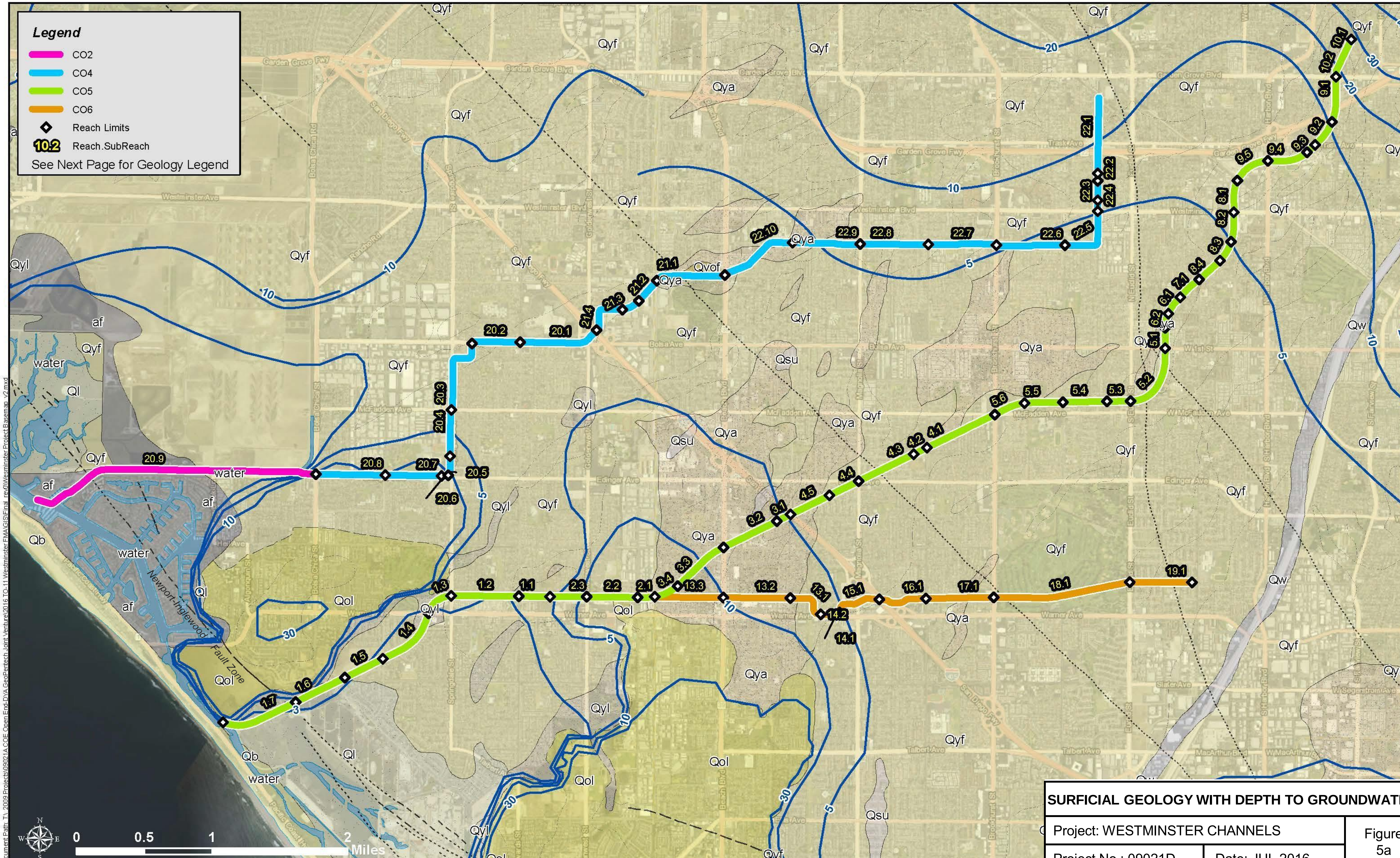

Legend Planned Bridge Modifications
Add Barrels / Upsize Barrels
Remove Bridge
Replace Bridge
Replace Bridge, Change from Trap. to Rect.
Replace Box with Barrels
Replace Piers with Barrels
Maintain Baseline Conditions
No Information Available
10.2 Reach.SubReach
C02
C04
C05
C06
0 0.5 1 2 Miles
USACE CHANNEL SYSTEM BRIDGE UPGRADES
Project: WESTMINSTER CHANNELS
Project No.: 09021D
Date: JUL 2016
Figure 4



SURFICIAL GEOLOGY WITH DEPTH TO GROUNDWATER

Project: WESTMINSTER CHANNELS		Figure 5a
Project No.: 09021D	Date: JUL 2016	

DIAZ•YOURMAN & ASSOCIATES Geotechnical Services GeoPentech KINNETIC LABORATORIES INCORPORATED

GEOLOGIC COMPILATION OF QUATERNARY SURFICIAL DEPOSITS IN SOUTHERN CALIFORNIA

LEGEND AND CORRELATION OF DERIVATIVE GEOLOGIC MAP UNITS

A Project for the Department of Water Resources by the California Geological Survey

Compiled from existing sources by
Trinda L. Bedrossian, CEG, Peter D. Roffers, and Cheryl A. Hayhurst, PG

July 2010

Geologic Map with Units Citation: Bedrossian, T.L., et al. (2012). Geologic Compilation of Quaternary Surficial Deposits in Southern California: California Geological Survey Special Report 217 (Revised), available at [http://www.conservation.ca.gov/cgs/fwgp/Pages/sr217.aspx#heading], accessed May 2016.

MAP UNITS

Late Holocene (Surficial Deposits)

- af — **Artificial Fill** - deposits of fill resulting from human construction, mining, or quarrying activities; includes engineered fill for buildings, roads, dams, airport runways, harbor facilities, and waste landfills
- Qsu — **Undifferentiated Surficial Deposits** - includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers
- Qls — **Landslide Deposits** - may include debris flows and older landslides of various earth material and movement types; unconsolidated to moderately well-consolidated
- Qb — **Beach Deposits** - unconsolidated marine beach sediments consisting mostly of fine- and medium-grained, well-sorted sand
- Qw — **Alluvial Wash Deposits** - unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand
- Qf — **Alluvial Fan Deposits** - unconsolidated boulders, cobbles, gravel, sand, and silt recently deposited where a river or stream issues from a confined valley or canyon; sediment typically deposited in a fan-shaped cone; gravelly sediment generally more dominant than sandy sediment
- Qa — **Alluvial Valley Deposits** - unconsolidated clay, silt, sand, and gravel recently deposited parallel to localized stream valleys and/or spread more regionally onto alluvial flats of larger river valleys; sandy sediment generally more dominant than gravelly sediment
- Qt — **Terrace Deposits** - includes marine and stream terrace deposits; marine deposits include slightly to moderately consolidated and bedded gravel and conglomerate, sand and sandstone, and silt and siltstone; river terrace deposits consist of unconsolidated thin- to thick-bedded gravel
- Ql — **Lacustrine, Playa, and Estuarine (Paralic) Deposits** - mostly unconsolidated fine-grained sand, silt, mud, and clay from fresh water (lacustrine) lakes, saline (playa) dry lakes that are periodically flooded, and estuaries; deposits may contain salt and other evaporites
- Qe — **Eolian and Dune Deposits** - unconsolidated, generally well-sorted wind-blown sand; may occur as dune forms or sheet sand

Holocene to Late Pleistocene (Surficial Deposits)

- Qyw — **Young Alluvial Wash Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected sandy and gravelly stream bed sediments in marginal parts of active and recently active washes and river channels
- Qyf — **Young Alluvial Fan Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
- Qya — **Young Alluvial Valley Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers
- Qyt — **Young Terrace Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected marine and stream terrace deposits
- Qyl — **Young Lacustrine, Playa, and Estuarine (Paralic) Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types
- Qye — **Young Eolian and Dune Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected wind-blown sands

Late to Middle Pleistocene (Surficial Deposits)

- Qow — **Old Alluvial Wash Deposits** - slightly to moderately consolidated, moderately dissected sand and gravel; typically elevated above modern washes
- Qof — **Old Alluvial Fan Deposits** - slightly to moderately consolidated, moderately dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
- Qoa — **Old Alluvial Valley Deposits** - slightly to moderately consolidated, moderately dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers
- Qot — **Old Terrace Deposits** - slightly to moderately consolidated, moderately dissected marine and stream terrace deposits
- Qol — **Old Lacustrine, Playa, and Estuarine (Paralic) Deposits** - slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types
- Qoe — **Old Eolian and Dune Deposits** - slightly to moderately consolidated, moderately dissected wind-blown sands

Middle to Early Pleistocene (Surficial Deposits)

- Qvow — **Very Old Alluvial Wash Deposits** - moderately to well-consolidated, highly dissected sand and gravel; typically elevated above modern washes
- Qvof — **Very Old Alluvial Fan Deposits** - moderately to well-consolidated, highly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
- Qvoa — **Very Old Alluvial Valley Deposits** - moderately to well-consolidated, highly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers; generally uplifted and deformed
- Qvot — **Very Old Terrace Deposits** - moderately to well-consolidated, highly dissected marine and stream terrace deposits
- Qvol — **Very Old Lacustrine, Playa, and Estuarine (Paralic) Deposits** - moderately to well-consolidated, highly dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types
- Qvoe — **Very Old Eolian and Dune Deposits** - moderately to well-consolidated, highly dissected wind-blown sands

SYMBOL EXPLANATION

[For geologic line symbols: lines are solid where location is accurate, long-dashed where location is approximate, short-dashed where location is inferred, dotted where location is concealed. Queries added where identity or existence may be questionable.]

Contacts
- Contact
- Gradational contact
- Reference contact -- Used to delineate geologic units that were mapped as separate units on the original source map, but are consolidated on this map

Fault -- Includes strike-slip, normal, reverse, oblique, and unspecified slip

Lineament

Folds -- Showing direction of plunge where appropriate
- Anticline
- Antiform or structural high
- Overturned anticline
- Syncline
- Synform or structural low
- Overturned syncline
- Monocline
- Generic fold

Scarps
- Scarp on fault
- Scarp on landslide
- Incised scarp of sedimentary contact
- Erosional scarp
- Fluvial terrace scarp
- Subsidence scarp

Outline of slip surface of landslide

Closed depression

Dike

Fissure

Dune crest

Former shoreline or marine limit

Stream

Spring

Wet area

Road

County boundary

State boundary

National boundary

CORRELATION OF MAP UNITS*

Quaternary Surficial Deposits (Alluvial Deposits: Alluvial Wash, Alluvial Fan, Alluvial Valley Deposits)

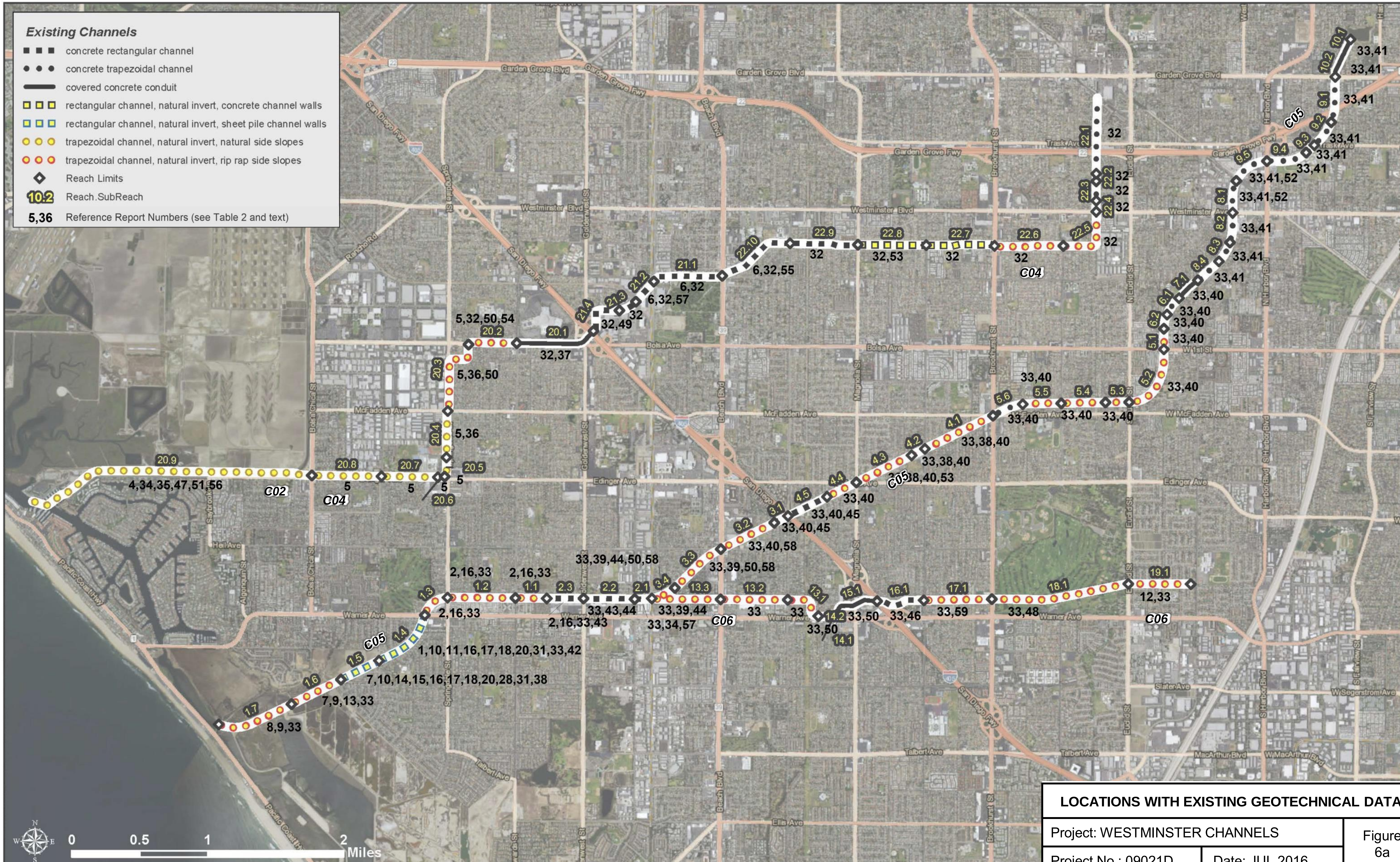
	Artificial Fill	Undiff. Surficial Deposits	Landslide Deposits	Beach Deposits	Alluvial Wash Deposits	Alluvial Fan Deposits	Alluvial Valley Deposits	Terrace Deposits	Lacustrine, Playa, and Estuarine Deposits	Eolian and Dune Deposits
Late Holocene	af	Qsu	Qls	Qb	Qw	Qf	Qa	Qt	Ql	Qe
Holocene to Late Pleistocene					Qyw	Qyf	Qya	Qyt	Qyl	Qye
Late to Middle Pleistocene					Qow	Qof	Qoa	Qot	Qol	Qoe
Middle to Early Pleistocene					Qvow	Qvof	Qvoa	Qvot	Qvol	Qvoe
	Coarse-grained	Fine-grained	Volcanic							
Quaternary (Bedrock)	Qss	Qsh	Qv							
Tertiary (Bedrock)	Tss	Tsh	Tv							
					Cretaceous and Pre-Cretaceous Metamorphic (sedimentary and volcanic)		Serpentinite (all ages)	Granitic Rocks (all ages)		
Mesozoic and Older (Bedrock)	Kss	Ksh	Kv		pKm		sp	gr		

* Boundaries of Quaternary units are gradational and time transgressive in a regional sense.

10 — Depth to Historically Highest Groundwater in Feet

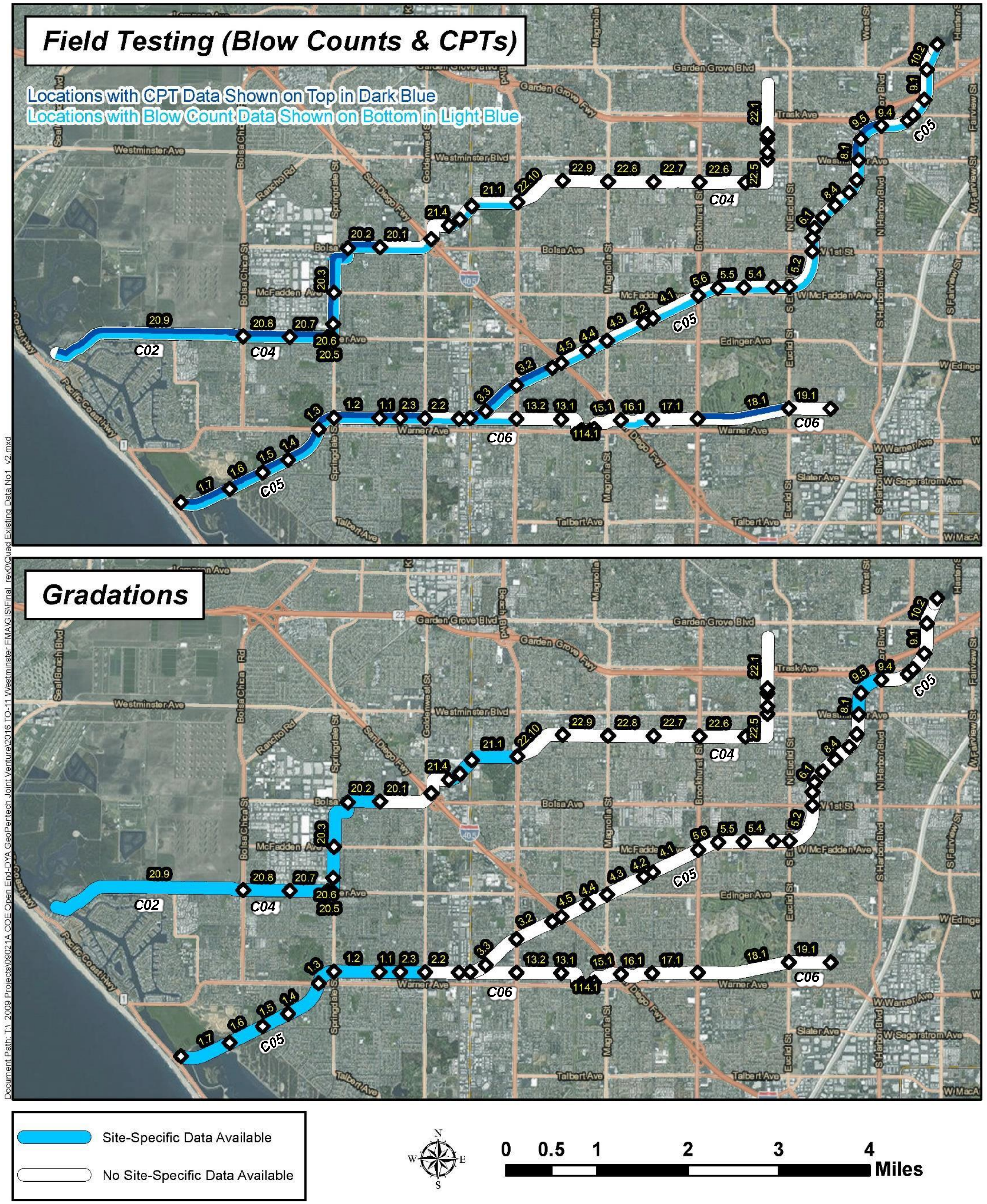
Groundwater Contours Citation: varies by quadrangle; example:
California Division of Mines and Geology (CDMG). 1998. Seismic Hazard Report for the Whittier 7.5-Minute Quadrangle, Los Angeles and Orange Counties, California: Seismic Hazard Zone Report 037, 57 pp., available at. [http://www.quake.ca.gov/gmaps/WH/regulatorymaps.htm], accessed May 2016.

SURFICIAL GEOLOGY LEGEND		
Project: WESTMINSTER CHANNELS		Figure 5b
Project No.: 09021D	Date: JUL 2016	



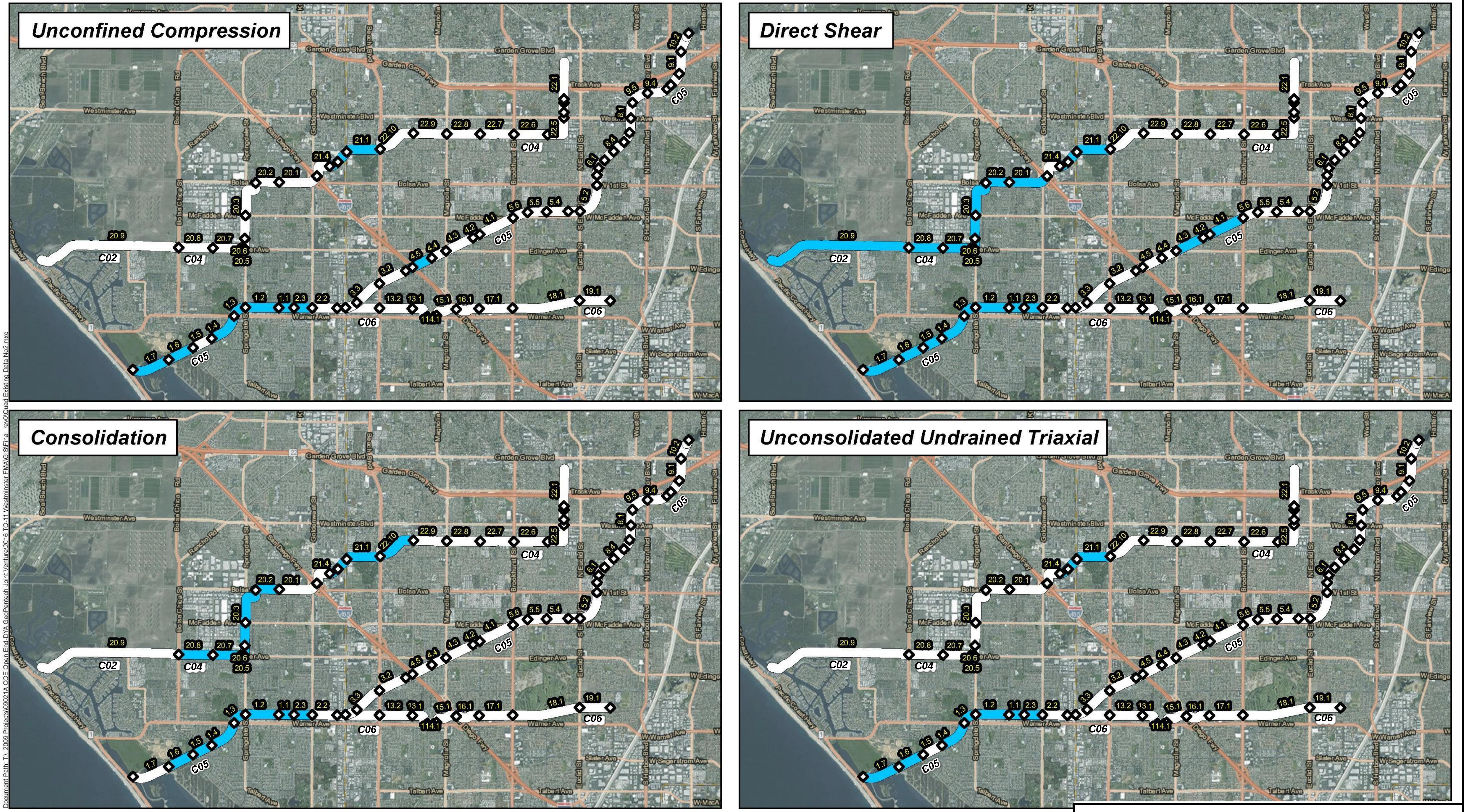
LOCATIONS WITH EXISTING GEOTECHNICAL DATA

Project: WESTMINSTER CHANNELS		Figure 6a
Project No.: 09021D	Date: JUL 2016	



LOCATIONS WITH EXISTING GEOTECHNICAL DATA

Project: WESTMINSTER CHANNELS		Figure 6b
Project No.: 09021D	Date: JUL 2016	

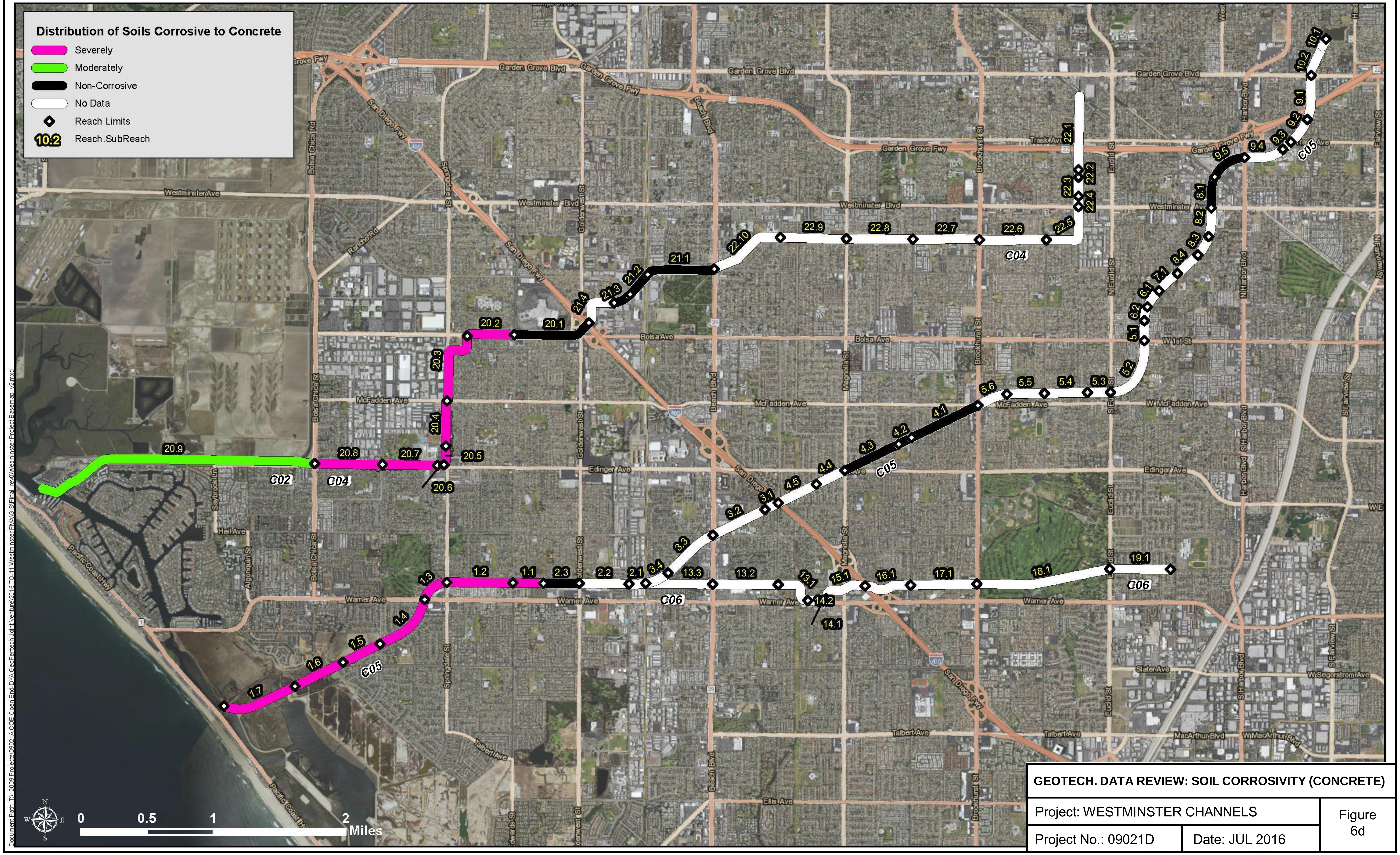


Site-Specific Data Available

No Site-Specific Data Available

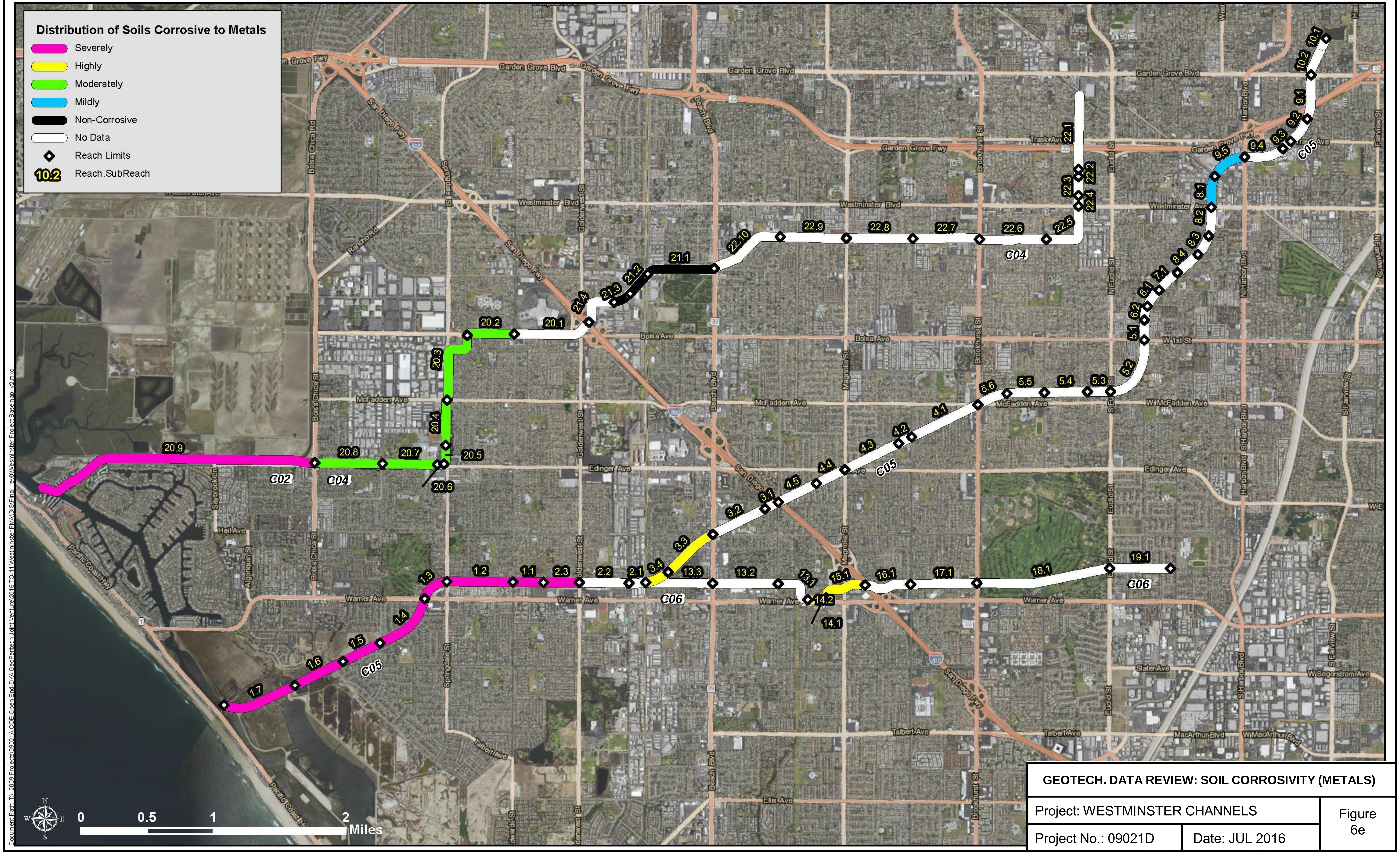
0 0.5 1 2 3 4 Miles

LOCATIONS WITH EXISTING GEOTECHNICAL DATA		
Project: WESTMINSTER CHANNELS		Figure 6c
Project No.: 09021D	Date: JUL 2016	



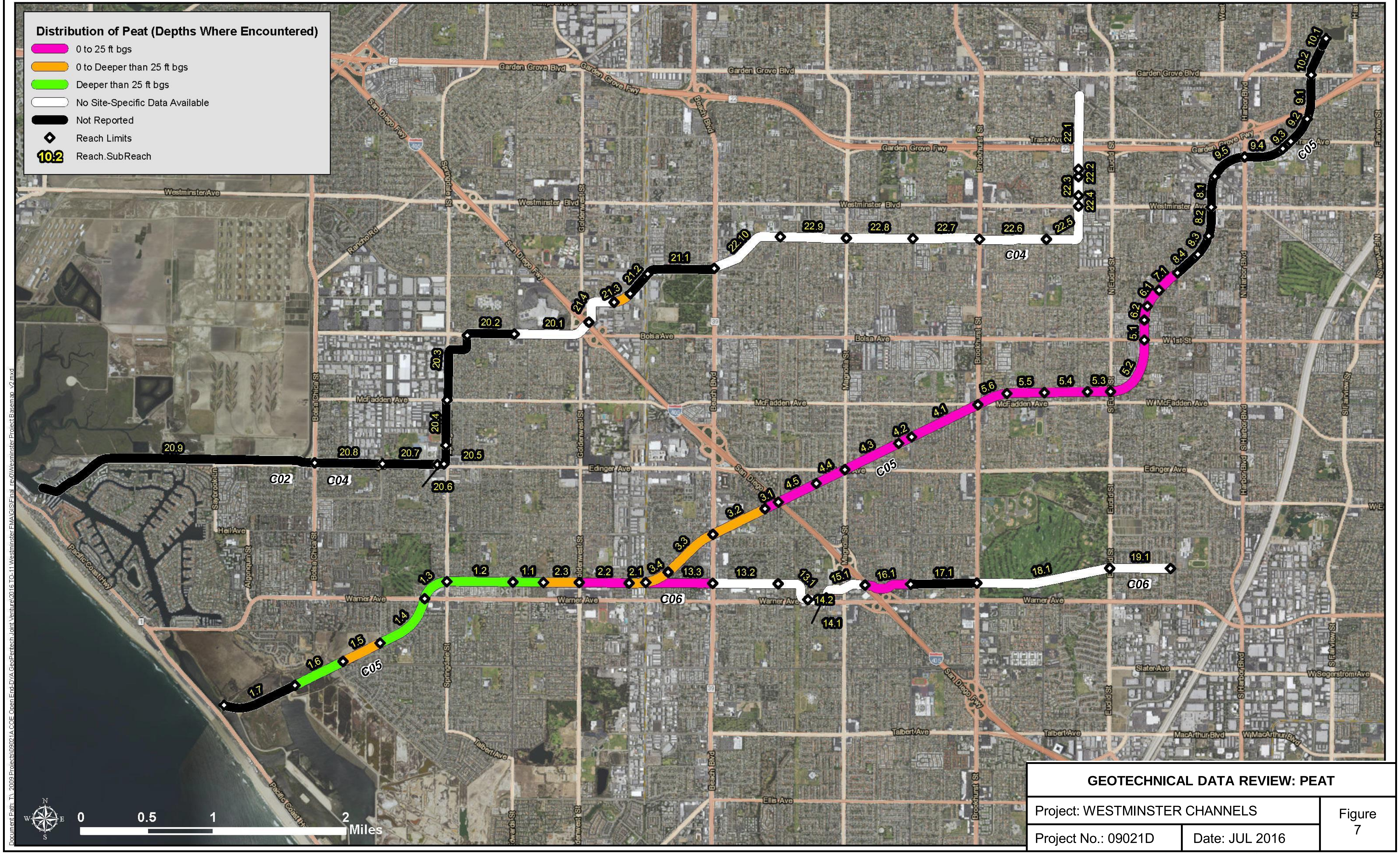
GEOTECH. DATA REVIEW: SOIL CORROSIVITY (CONCRETE)

Project: WESTMINSTER CHANNELS		Figure 6d
Project No.: 09021D	Date: JUL 2016	



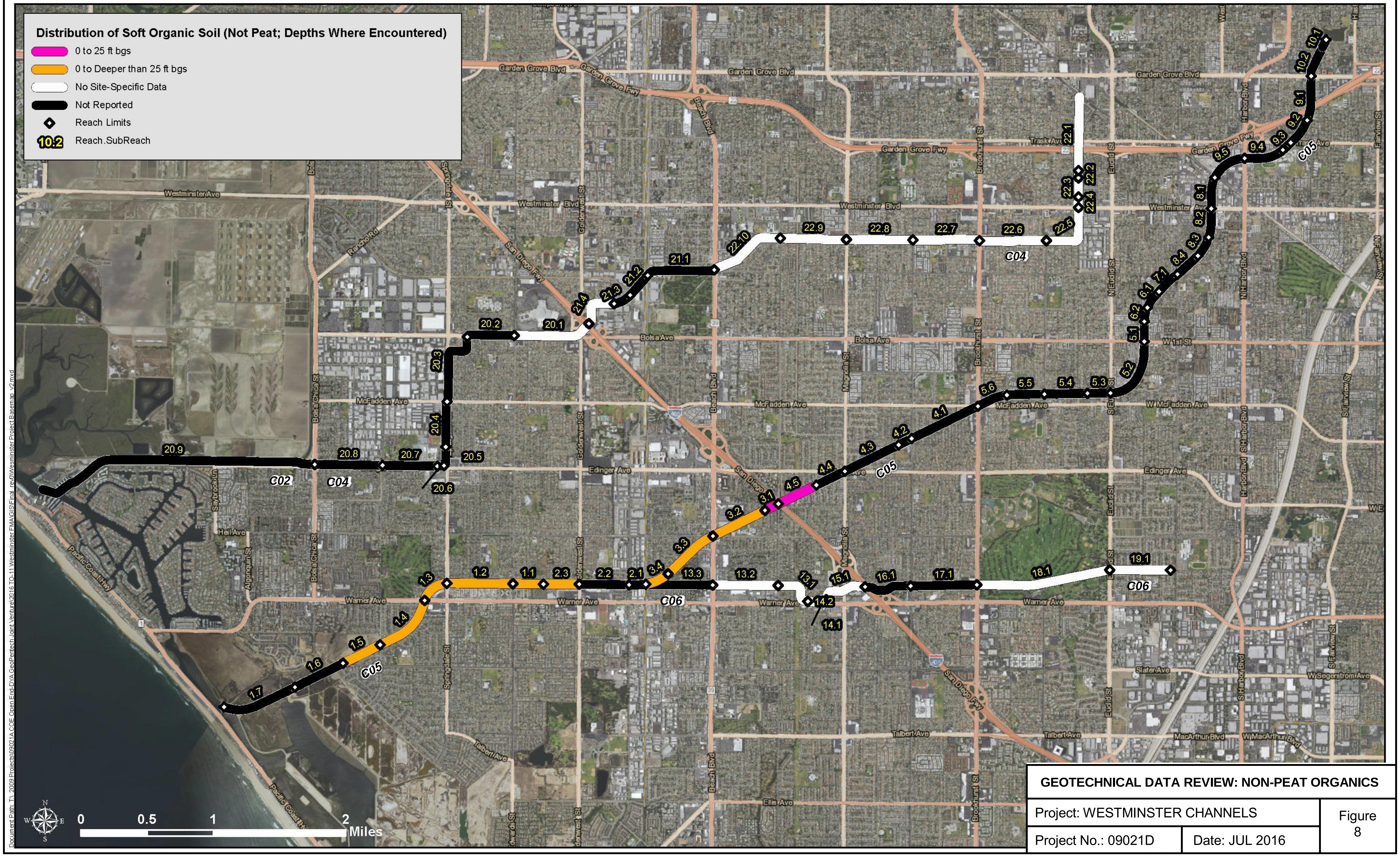
GEOTECH. DATA REVIEW: SOIL CORROSIVITY (METALS)

Project: WESTMINSTER CHANNELS		Figure 6e
Project No.: 09021D	Date: JUL 2016	



GEOTECHNICAL DATA REVIEW: PEAT

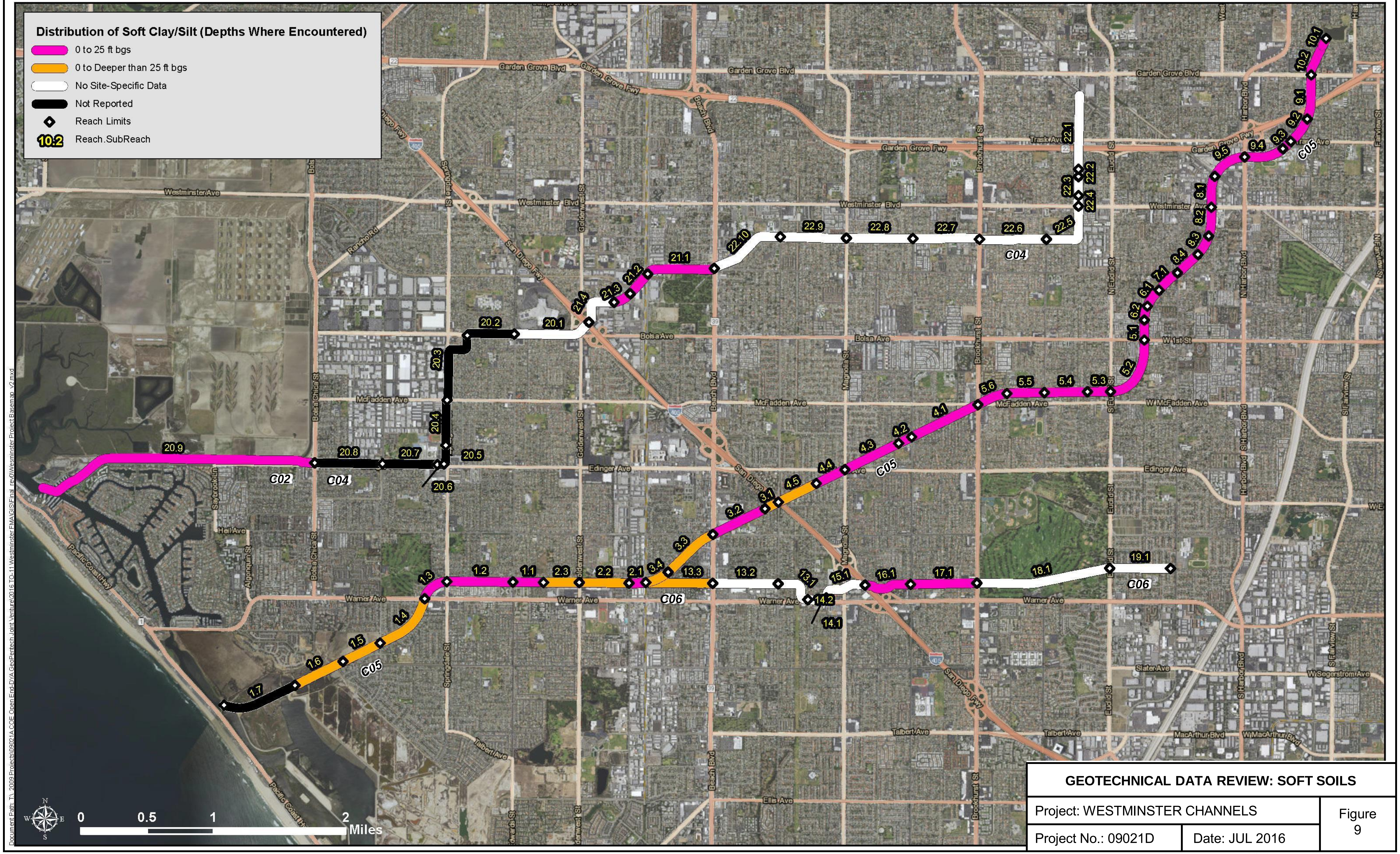
Project: WESTMINSTER CHANNELS	Figure 7
Project No.: 09021D	Date: JUL 2016



GEOTECHNICAL DATA REVIEW: NON-PEAT ORGANICS

Project: WESTMINSTER CHANNELS		Figure 8
Project No.: 09021D	Date: JUL 2016	

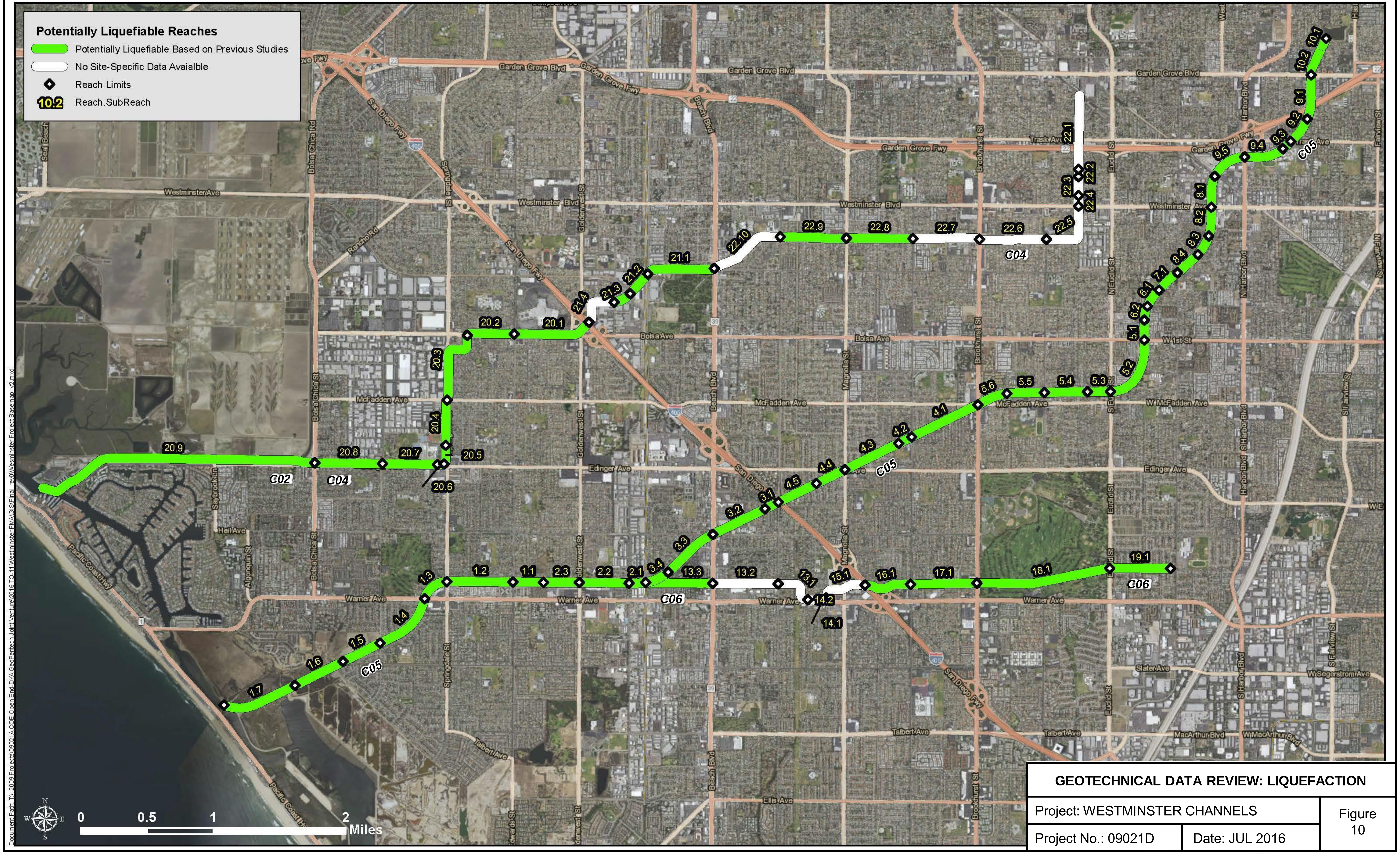
DIAZ YOURMAN & ASSOCIATES Geotechnical Services · GeoPentech · KINNETIC LABORATORIES INCORPORATED



GEOTECHNICAL DATA REVIEW: SOFT SOILS

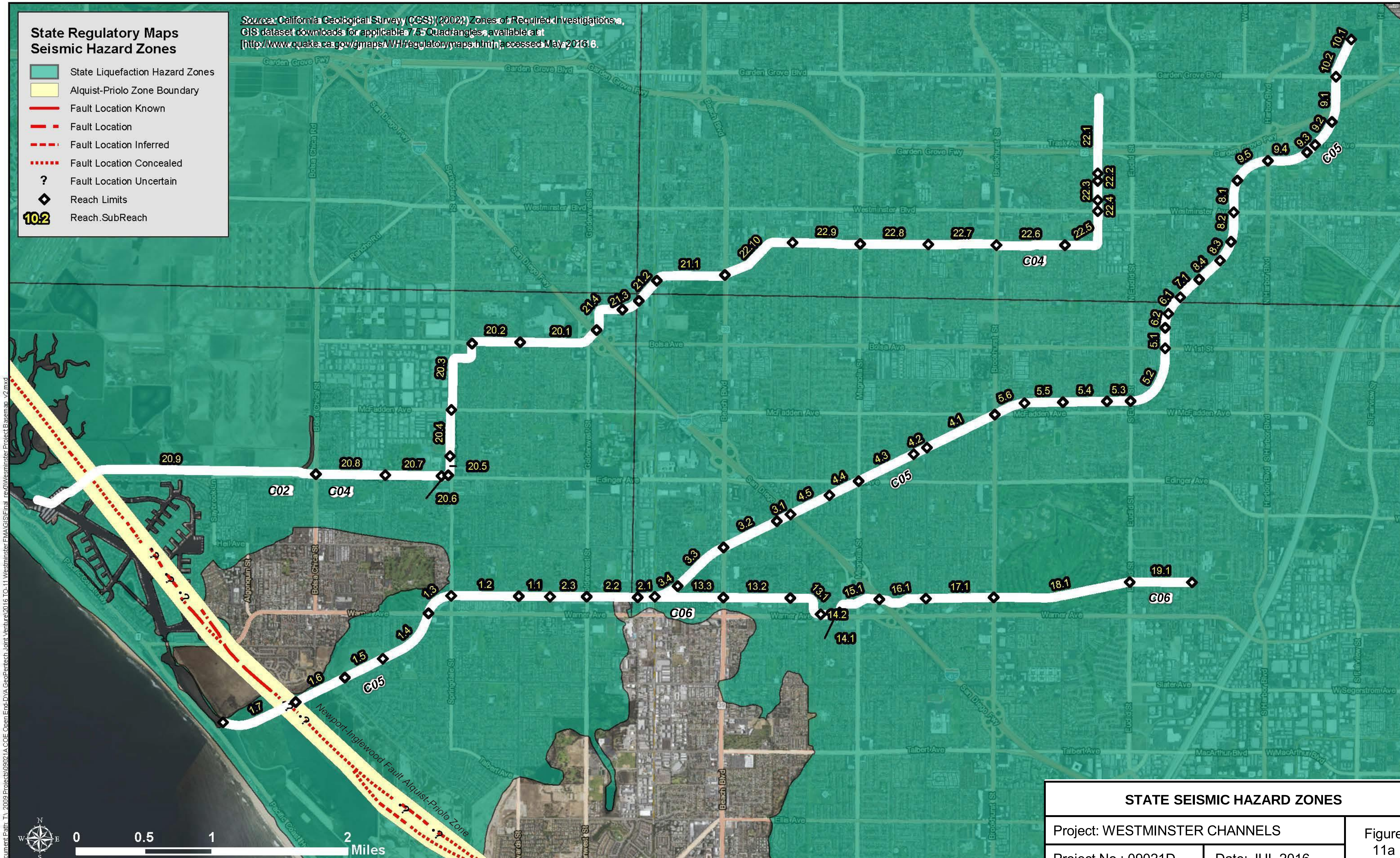
Project: WESTMINSTER CHANNELS		Figure 9
Project No.: 09021D	Date: JUL 2016	

DIAZ•YOURMAN & ASSOCIATES Geotechnical Services
GeoPentech
KINNETIC LABORATORIES INCORPORATED



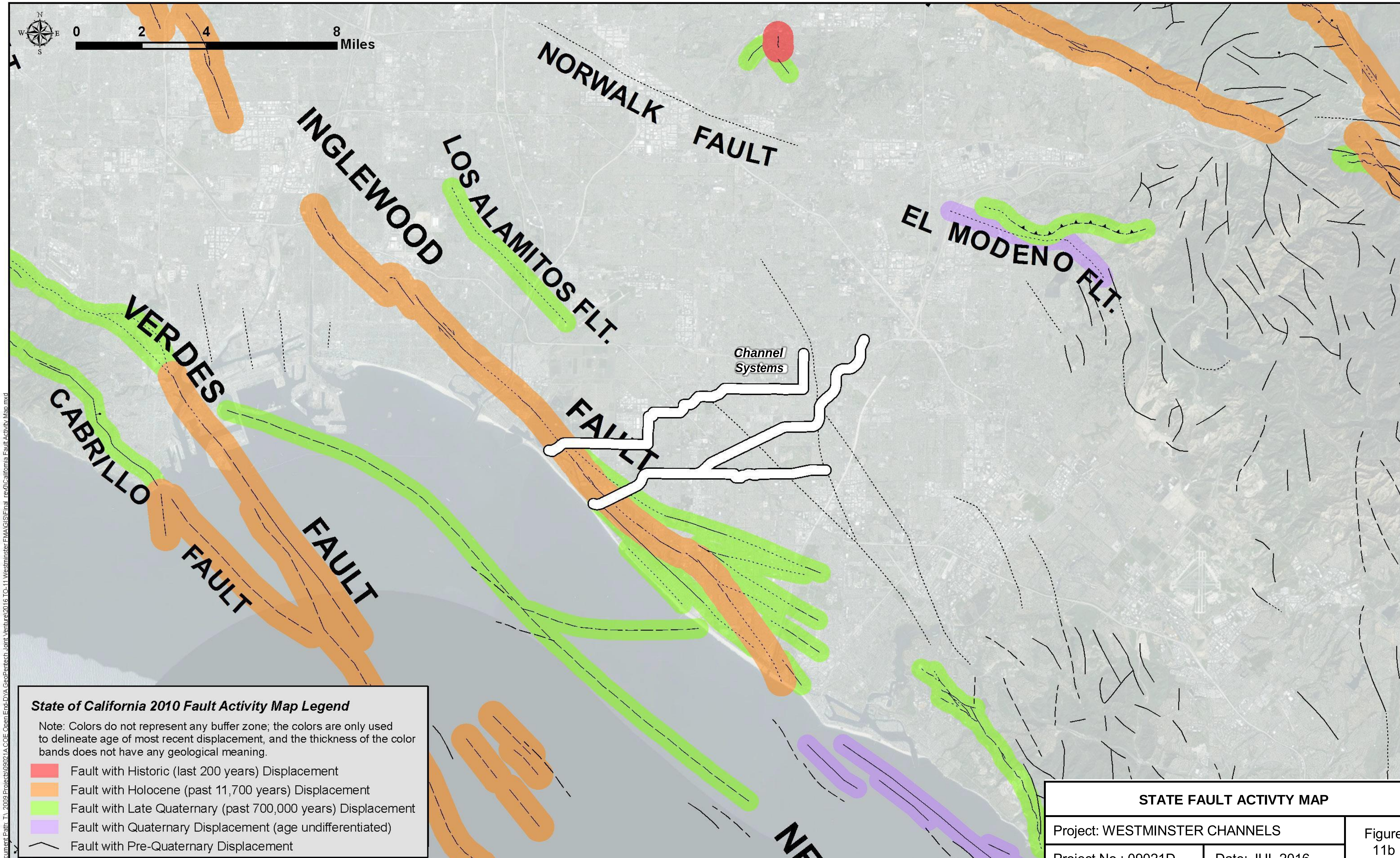
GEOTECHNICAL DATA REVIEW: LIQUEFACTION

Project: WESTMINSTER CHANNELS		Figure 10
Project No.: 09021D	Date: JUL 2016	

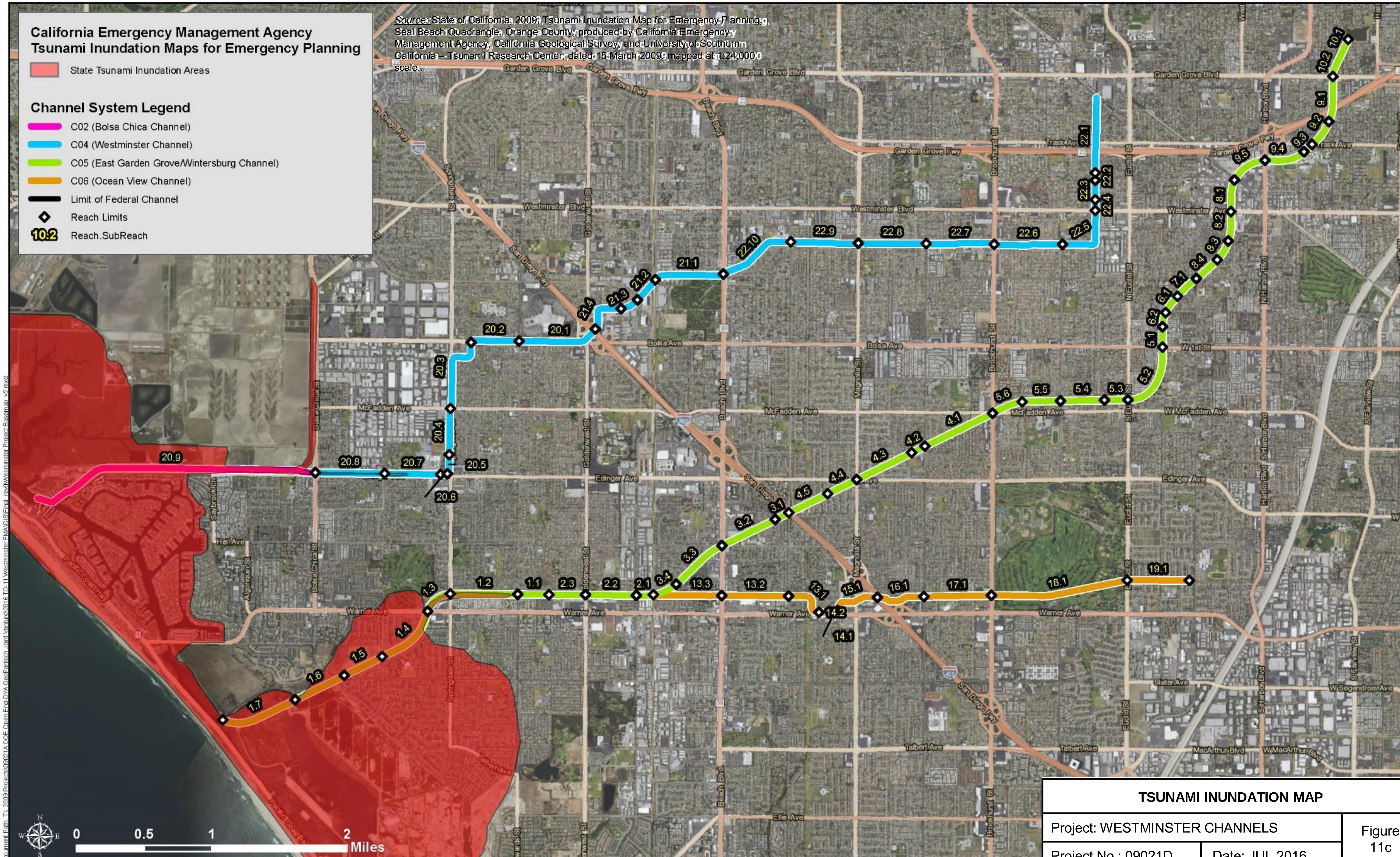


STATE SEISMIC HAZARD ZONES

Project: WESTMINSTER CHANNELS		Figure 11a
Project No.: 09021D	Date: JUL 2016	

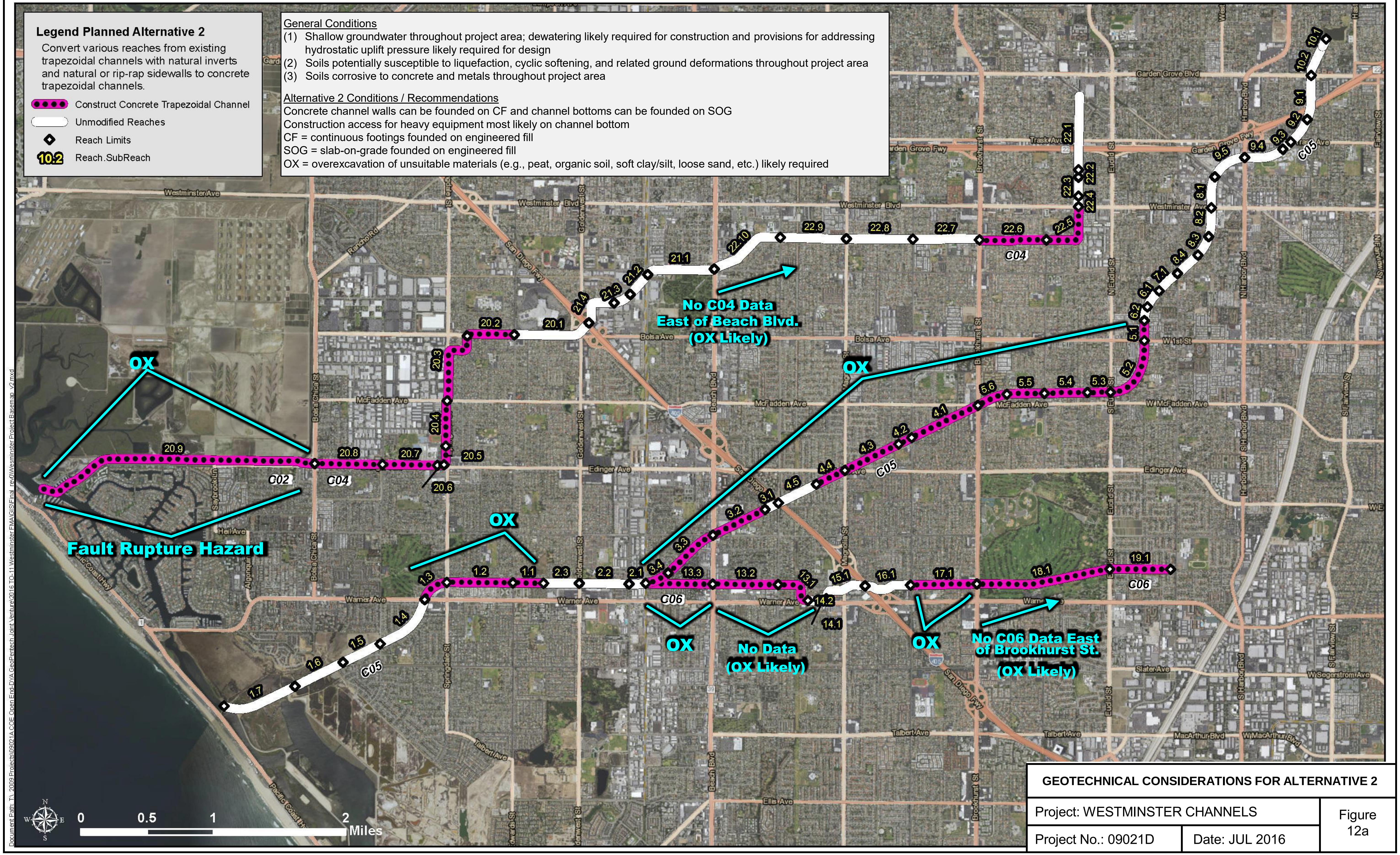


STATE FAULT ACTIVTY MAP		
Project: WESTMINSTER CHANNELS		Figure 11b
Project No.: 09021D	Date: JUL 2016	



TSUNAMI INUNDATION MAP

Project: WESTMINSTER CHANNELS		Figure 11c
Project No.: 09021D	Date: JUL 2016	



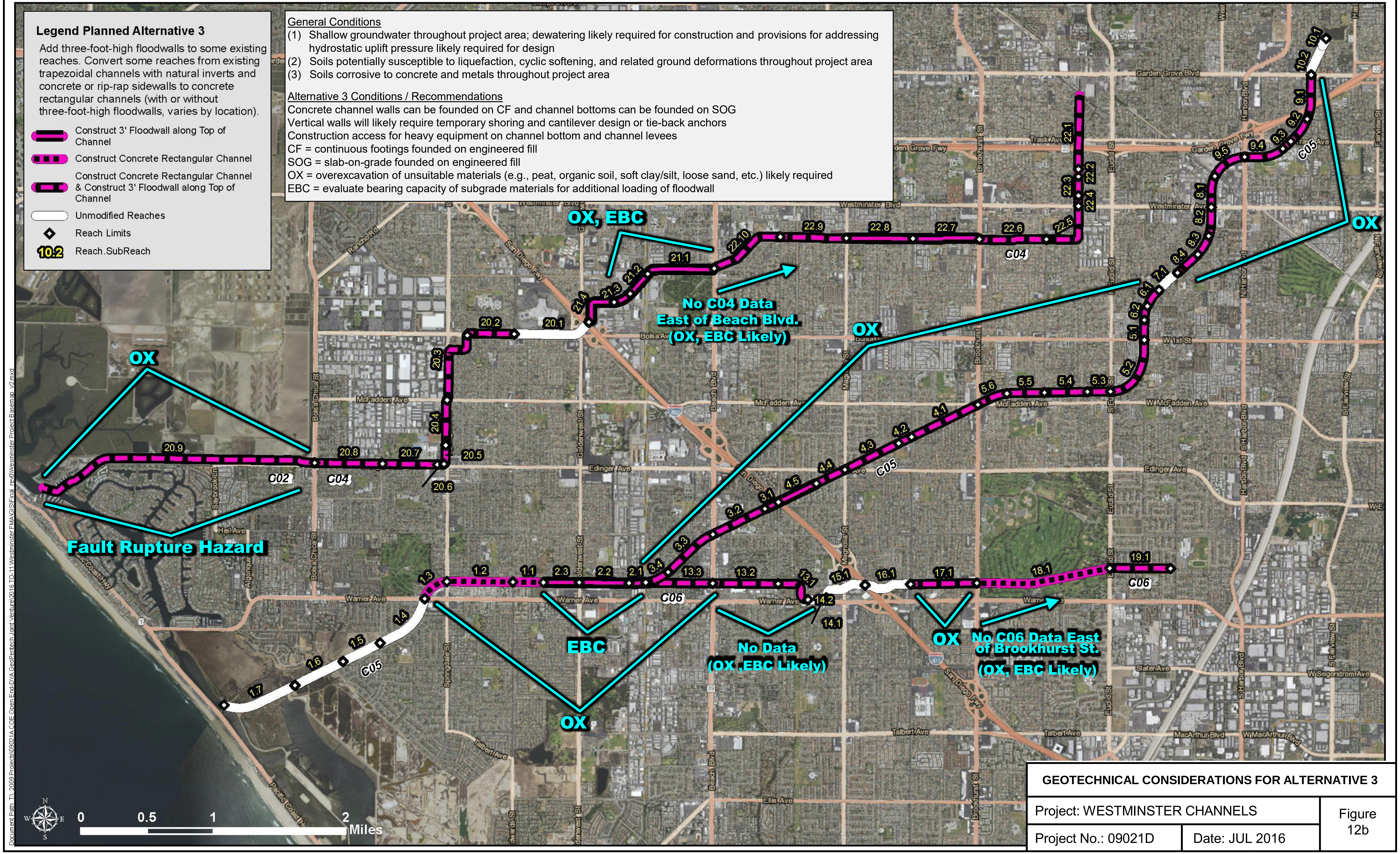
GEOTECHNICAL CONSIDERATIONS FOR ALTERNATIVE 2

Project: WESTMINSTER CHANNELS

Project No.: 09021D

Date: JUL 2016

Figure 12a



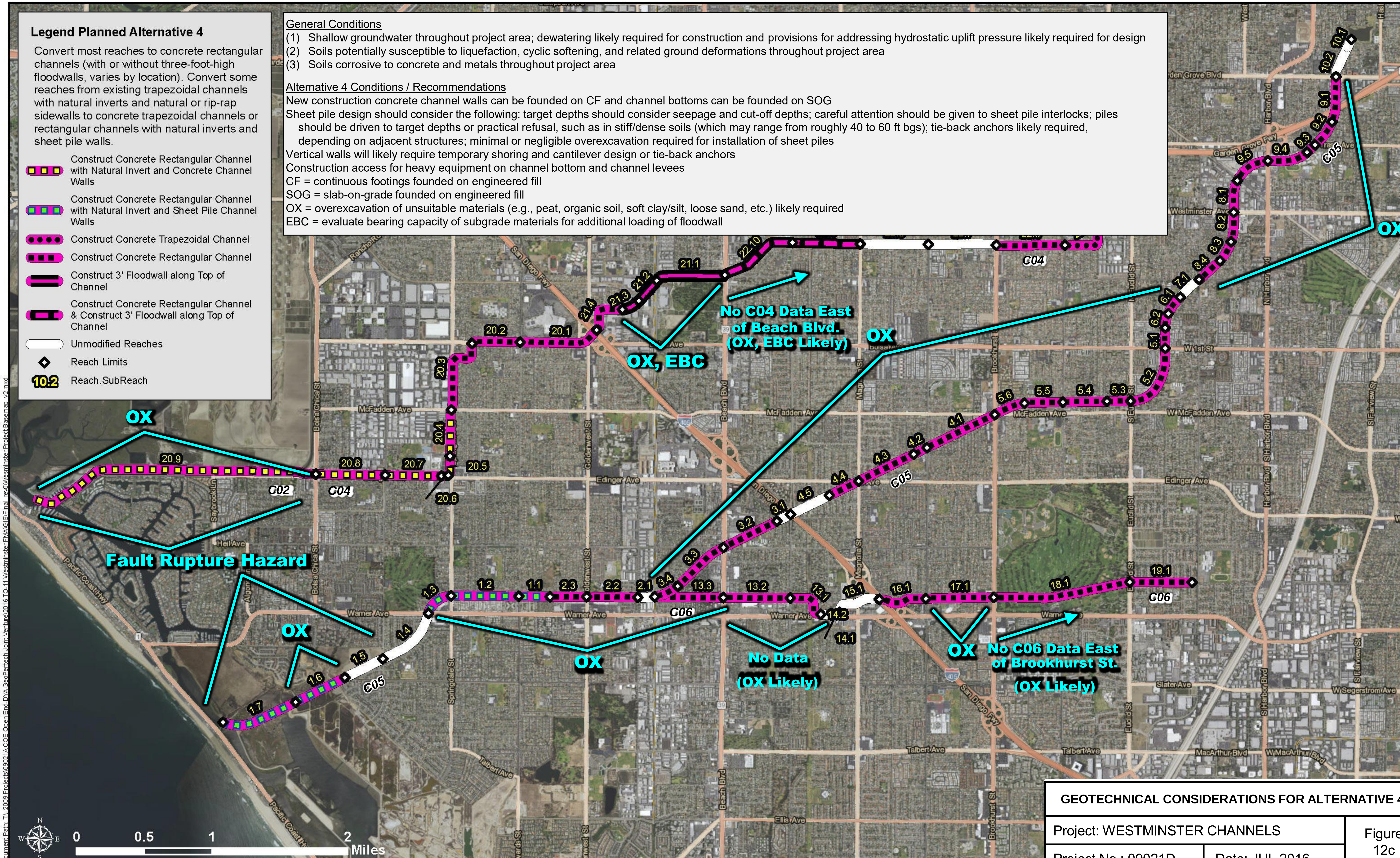
GEOTECHNICAL CONSIDERATIONS FOR ALTERNATIVE 3

Project: WESTMINSTER CHANNELS

Project No.: 09021D

Date: JUL 2016

Figure 12b



General Conditions
(1) Shallow groundwater throughout project area; dewatering likely required for construction and provisions for addressing hydrostatic uplift pressure likely required for design
(2) Soils potentially susceptible to liquefaction, cyclic softening, and related ground deformations throughout project area
(3) Soils corrosive to concrete and metals throughout project area

Alternative 4 Conditions / Recommendations
New construction concrete channel walls can be founded on CF and channel bottoms can be founded on SOG
Sheet pile design should consider the following: target depths should consider seepage and cut-off depths; careful attention should be given to sheet pile interlocks; piles should be driven to target depths or practical refusal, such as in stiff/dense soils (which may range from roughly 40 to 60 ft bgs); tie-back anchors likely required, depending on adjacent structures; minimal or negligible overexcavation required for installation of sheet piles
Vertical walls will likely require temporary shoring and cantilever design or tie-back anchors
Construction access for heavy equipment on channel bottom and channel levees
CF = continuous footings founded on engineered fill
SOG = slab-on-grade founded on engineered fill
OX = overexcavation of unsuitable materials (e.g., peat, organic soil, soft clay/silt, loose sand, etc.) likely required
EBC = evaluate bearing capacity of subgrade materials for additional loading of floodwall

GEOTECHNICAL CONSIDERATIONS FOR ALTERNATIVE 4

Project: WESTMINSTER CHANNELS		Figure 12c
Project No.: 09021D	Date: JUL 2016	

Legend Planned Alternative 5

Convert various reaches from existing trapezoidal channels with natural inverts and natural or rip-rap sidewalls to concrete trapezoidal channels. Construct new detention basin in Mile Square Park along C06 channel and new buried conduit under Ward Street, connecting C05 channel to new basin.

- Construct Concrete Trapezoidal Channel
- Buried Conduit
- Detention Basin
- Unmodified Reaches
- Reach Limits
- 10.2 Reach.SubReach

General Conditions

(1) Shallow groundwater throughout project area; dewatering likely required for construction and provisions for addressing hydrostatic uplift pressure likely required for design

(2) Soils potentially susceptible to liquefaction, cyclic softening, and related ground deformations throughout project area

(3) Soils corrosive to concrete and metals throughout project area

Alternative 5 Conditions / Recommendations

New construction concrete channel walls can be founded on CF and channel bottoms can be founded on SOG

Buried conduit likely to be construction as cut-and-cover; temporary shoring and suitable backfill materials required

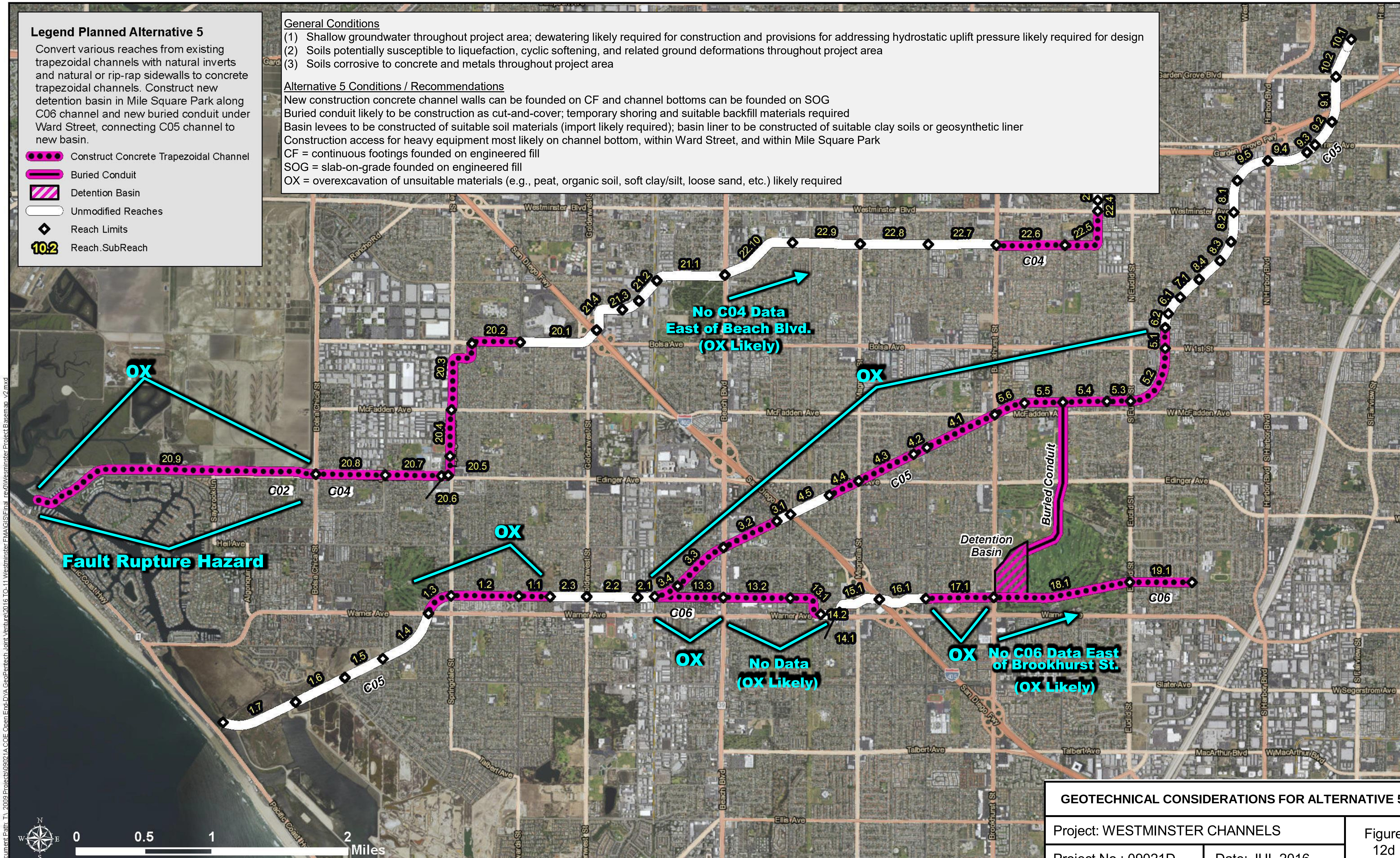
Basin levees to be constructed of suitable soil materials (import likely required); basin liner to be constructed of suitable clay soils or geosynthetic liner

Construction access for heavy equipment most likely on channel bottom, within Ward Street, and within Mile Square Park

CF = continuous footings founded on engineered fill

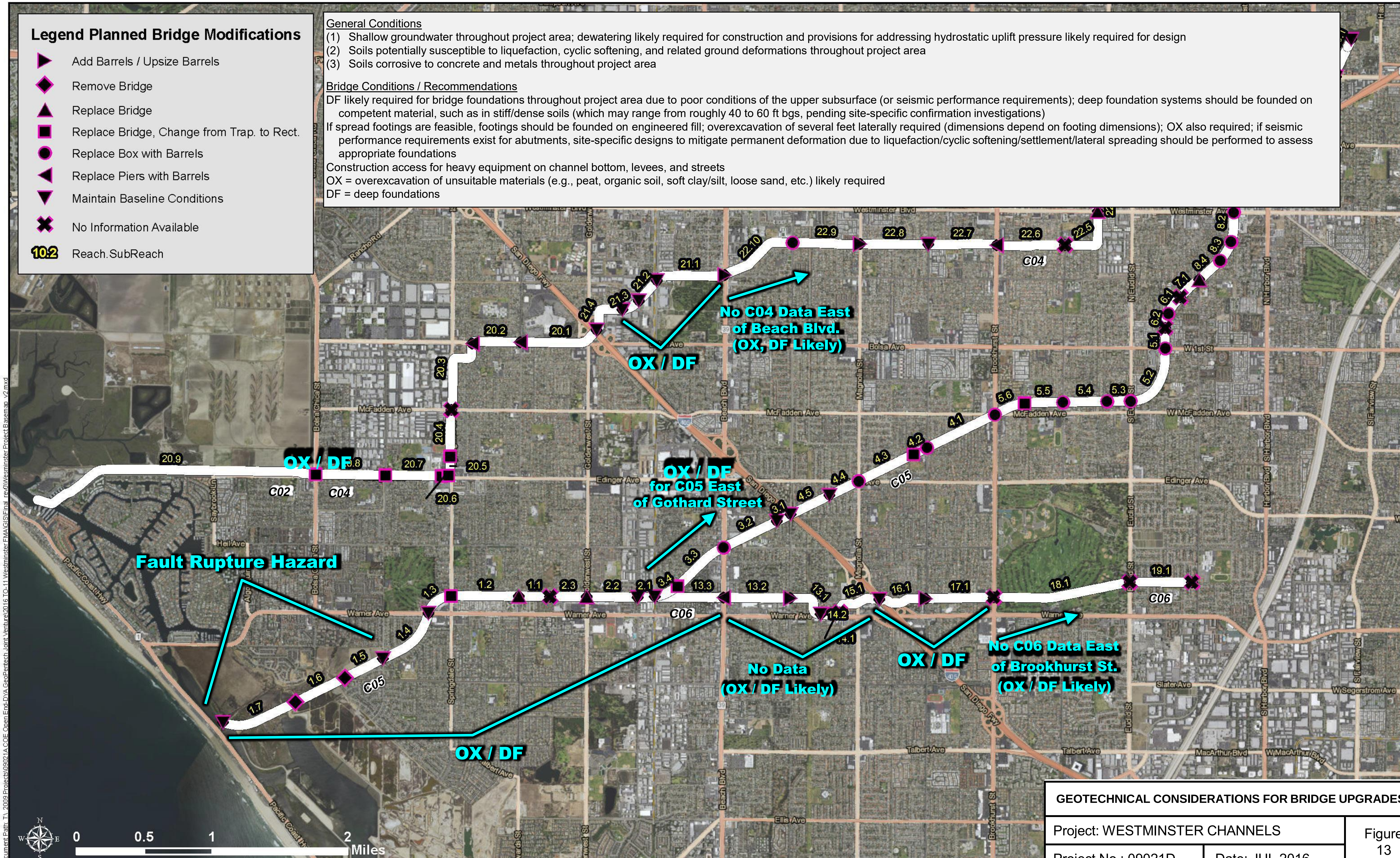
SOG = slab-on-grade founded on engineered fill

OX = overexcavation of unsuitable materials (e.g., peat, organic soil, soft clay/silt, loose sand, etc.) likely required



GEOTECHNICAL CONSIDERATIONS FOR ALTERNATIVE 5		
Project: WESTMINSTER CHANNELS		Figure 12d
Project No.: 09021D	Date: JUL 2016	

Document Path: T:\2009 Projects\09021A COE Open End-DVA GeoPentech Joint Venture\2016 TO-11 Westminster FMA\GIS\Final_rev\Westminster Project Basemap_v2.mxd



General Conditions
(1) Shallow groundwater throughout project area; dewatering likely required for construction and provisions for addressing hydrostatic uplift pressure likely required for design
(2) Soils potentially susceptible to liquefaction, cyclic softening, and related ground deformations throughout project area
(3) Soils corrosive to concrete and metals throughout project area

Bridge Conditions / Recommendations
DF likely required for bridge foundations throughout project area due to poor conditions of the upper subsurface (or seismic performance requirements); deep foundation systems should be founded on competent material, such as in stiff/dense soils (which may range from roughly 40 to 60 ft bgs, pending site-specific confirmation investigations)
If spread footings are feasible, footings should be founded on engineered fill; overexcavation of several feet laterally required (dimensions depend on footing dimensions); OX also required; if seismic performance requirements exist for abutments, site-specific designs to mitigate permanent deformation due to liquefaction/cyclic softening/settlement/lateral spreading should be performed to assess appropriate foundations
Construction access for heavy equipment on channel bottom, levees, and streets
OX = overexcavation of unsuitable materials (e.g., peat, organic soil, soft clay/silt, loose sand, etc.) likely required
DF = deep foundations

GEOTECHNICAL CONSIDERATIONS FOR BRIDGE UPGRADES

Project: WESTMINSTER CHANNELS		Figure 13
Project No.: 09021D	Date: JUL 2016	

ATTACHMENTS

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	1	2	3
Subreach No.	1.4	1.1, 1.2, 1.3, 2.3	N/A
Title	Final Geotechnical Investigation Report, East Garden Grove-Wintersburg Channel Improvements from Graham Street to Warner Avenue (Station 75+00 to 100+00), Huntington Beach, Orange County, California, 349 pp.	Final Geotechnical Investigation Report, East Garden Grove-Wintersburg Channel Improvements from Warner Avenue to Upstream of Edwards Street (Station 101+00 to 151+25), Huntington Beach, Orange County, California, 717 pp.	Westminster Feasibility Study Preliminary Draft Baseline Conditions Report, Appendix F, Geotechnical, 21 pp.
Author	Earth Mechanics, Inc.	Earth Mechanics, Inc.	USACOE
Date	10/15/2009	10/15/2009	5/2007
Field Exploration	- 4 borings (drill method unknown), up to 80 ft deep - 26 CPTs, up to 80 ft deep	- 8 borings (rotary wash), up to 80 ft deep - 51 CPTs, up to 80 ft deep	Not Applicable or No Data
Field / Laboratory Tests	Blow Counts; Pocket Penetrometer; Moisture Content; Unit Weight; Sieve; Sand Equivalent; Atterberg Limits; Corrosivity; Unconfined Compression; Direct Shear; Consolidation; Unconsolidated Undrained Triaxial	Blow Counts; Pore Presssure Dissipation Test; Moisture Content; Unit Weight; Sieve; #200 Wash; Sand Equivalent; Atterberg Limits; Max. Dry Density/Optimum Moisture; Corrosivity; Unconfined Compression; Direct Shear; Consolidation; Unconsolidated Undrained Triaxial	Not Applicable or No Data
Maps	Plan sheets with exploration locations	Plan sheets with exploration locations	- Groundwater contours for northern Orange County
Sections / Profiles	Boring logs; CPT logs; 5 schematic / interpreted cross sections	Boring logs; CPT logs; 9 schematic / interpreted cross sections	Not Applicable or No Data
Geotechnical Analyses	Liquefaction analysis using CLiq software; seismically-induced settlement analysis using CLiq software	Liquefaction analysis using CLiq software; seismically-induced settlement analysis using CLiq software; slope stability analysis	Not Applicable or No Data
Geotechnical Design Parameters	Modulus of subgrade reaction	Lateral earth pressures; modulus of subgrade reaction	Not Applicable or No Data
Depth to Groundwater	13 ft to 24 ft bgs	12 ft to 30 ft bgs	- Avg. depth ~10 ft bgs - Depth ranges from 0 ft to 19 ft bgs in most areas, not encountered in eastern reaches
Soil Corrosivity	- Moderately to highly corrosive to metals - Non-corrosive to concrete	- Moderately to highly corrosive to metals - Non-corrosive to concrete	Not Applicable or No Data
Liquefaction Potential	- Report states soils in upper 60 ft are potentially liquefiable	- Report states soils in upper 60 ft are potentially liquefiable	Not Applicable or No Data
Peat Layers	- Top of peat generally 31 ft to 35 ft bgs; thickness varies from approx. 1 ft to 5 ft - Top of soft organic clay varies from 15 ft to 36 ft bgs; thickness varies from approx. 4 ft to 20 ft; typically with interbedded layers of medium dense sand, silt, and/or sand with silt	- Top of peat varies from 25 ft to 31 ft bgs; thickness approx. 4 ft to 5 ft - Top of soft organic clay varies from 14 ft to 18 ft bgs; thickness varies from approx. 1 ft to 5 ft, often interbedded with layers of soft elastic silt and/or loose to medium dense sand with silt	Not Applicable or No Data
Soft Clay / Silt Layers	Not Applicable or No Data	Top of soft clay/silt varies from 14 ft to 18 ft bgs; thickness varies from approx. 2 ft to 10 ft, typically interbedded	Not Applicable or No Data
Additional Comments	Not Applicable or No Data	- Seismically-induced settlement generally 0.4 inch to 3.5 inches (CPT-08-50 shows 8.5 inches) - Slope stability static factors of safety ≥ 1.5; pseudo-static factors of safety < 1	- Hundreds of geotechnical subsurface explorations have been completed throughout the system in the last ~50 yrs; there should be sufficient information available for feasibility study/design

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	4	5	6
Subreach No.	20.9	20.2, 20.3, 20.4, 20.5, 20.6, 20.7, 20.8	21.1, 21.2, 22.10
Title	Project Report, Bolsa Chica Channel from Huntington Harbor Outlet to Cerritos Avenue, 270 pp.	Geotechnical Evaluation, Westminster Channel Improvements, Huntington Beach, California, 189 pp.	Final Geotechnical Investigation Report, Westminster Channel Improvements (C04) from Hoover Street to Beach Boulevard, County of Orange, California, 68 pp.
Author	Robert Bein, William Frost & Associates	Ninyo & Moore	Earth Mechanics, Inc.
Date	6/1983	5/24/2011	1/11/2007
Field Exploration	- 8 borings (drill method unknown), up to 25 ft deep	- 12 borings (hollow-stem auger), up to 51 ft deep - 4 CPTs, up to 50 ft deep	- 7 borings (hollow-stem auger), up to 55 ft deep
Field / Laboratory Tests	Moisture Content; Unit Weight; Sieve; Direct Shear	Blow Counts; Moisture Content; Unit Weight; Sieve; #200 Wash; Sand Equivalent; Atterberg Limits; R-value; Corrosivity; Direct Shear; Consolidation	Blow Counts; Moisture Content; Unit Weight; Sieve; #200 Wash; Sand Equivalent; Atterberg Limits; Max. Dry Density/Optimum Moisture; Corrosivity; Unconfined Compression; Direct Shear; Consolidation; Unconsolidated Undrained Triaxial
Maps	Map with exploration locations	Plan sheets with exploration locations	Plan sheets with exploration locations
Sections / Profiles	Boring logs (outdated quality)	Boring logs; CPT logs	Boring logs
Geotechnical Analyses	Not Applicable or No Data	Liquefaction analysis using LiquefyPro software; seismically-induced settlement analysis; lateral spreading analysis; slope stability analysis; pile analysis using LPile software	Liquefaction analysis; seismically-induced settlement analysis
Geotechnical Design Parameters	Not Applicable or No Data	Lateral earth pressures; modulus of subgrade reaction	Lateral earth pressures; bearing capacity
Depth to Groundwater	8.5 ft to 16 ft bgs	13 ft to 22 ft bgs	8 ft to 28 ft bgs
Soil Corrosivity	Not Applicable or No Data	- Corrosive to metals - Severely corrosive to concrete	Non-corrosive
Liquefaction Potential	Not Applicable or No Data	- Report states soils in upper 40 ft are potentially liquefiable	- Report states some site soils are liquefiable - Boring logs show low blow count cohesionless soils approx. 10 ft to 40 ft bgs, locally interbedded with denser/stiffer soils
Peat Layers	Unknown, not distinguished on logs	Not encountered	Not encountered
Soft Clay / Silt Layers	Unknown, not distinguished on logs	Not encountered	Top of soft clay and/or soft silt generally 10 ft to 20 ft bgs, thickness varies significantly from approx. 5 ft to 20 ft; locally top as shallow as 2 ft bgs with thickness up to 28 ft; often interbedded with layers of loose to medium dense sand and sand with silt
Additional Comments	- Soils are unconsolidated and therefore subject to consolidation during construction due to dewatering and loading - Settlement due to consolidation is possible	- Seismically-induced settlement up to 2 inches, differential settlement up to 1.5 inches/40 ft - Soft clays are present and susceptible to consolidation settlement - Generally, shallow (open-face) soils are susceptible to lateral spreading with displacements of up to 15 ft - Slope stability static and pseudo-static factors of safety ≥ 1	- Seismically-induced settlement up to 4 inches - No lateral spreading hazard - Report states no liquefaction mitigation needed; however, reviewer comments suggest the recommendation may have been subsequently modified

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	7	8	9
Subreach No.	1.5, 1.6	1.7	1.6, 1.7
Title	Plans for Construction of East Garden Grove - Wintersburg Channel North Levee Emergency Project from 3800 feet Downstream of Graham St. to Graham St., 22 pp.	East Garden Grove - Wintersburg Channel, OCFCD Facility C05, Slope Stability Analysis of the North Levee Downstream of the Oil Bridge, 43 pp.	Geotechnical Investigation Report for East Garden Grove - Wintersburg Channel (OCFCD Facility C05) Improvements Phase 1, Proposed Sheet Pile Buttress Support from STA. 34+00 to STA. 53-16, City of Huntington Beach, Orange County, California, 163 pp.
Author	Orange County Resources and Development Management Dept.	Orange County Flood Control District	URS Corpoation
Date	10/15/2007	9/23/2008	1/18/2011
Field Exploration	- 1 Boring (rotary wash), 40 ft deep, from MACTEC 5/23/2003 report - (Repeats 9 borings from Report No. 16)	Not Applicable or No Data	- 3 borings (rotary wash), up to 66 ft deep - 5 CPTs, up to 65 ft deep
Field / Laboratory Tests	Blow Counts; Moisture Content; Unit Weight; Corrosivity; Direct Shear	Moisture Content; Unit Weight; Direct Shear	Blow Counts; Moisture Content; Unit Weight; Sieve; #200 Wash; Sand Equivalent; Atterberg Limits; Max. Dry Density/Optimum Moisture; Permeability; Corrosivity; Unconfined Compression; Direct Shear; Unconsolidated Undrained Triaxial
Maps	No map for MACTEC 5/23/2003 report; Map with exploration locations from Report No. 16	Not Applicable or No Data	Map with exploration locations
Sections / Profiles	Boring log (page 1 only) for MACTEC 5/23/2003 report boring; 1 schematic / interpreted cross section from MACTEC 5/23/2003 report; Repeats summary of borings from Report No. 16, actual logs not included	Not Applicable or No Data	Boring logs; CPT logs; 1 schematic / interpreted cross section; 1 cross section showing potentially liquefiable zones
Geotechnical Analyses	Not Applicable or No Data	Slope stability analysis using GSTABL7.2 software	Liquefaction analysis using CPet-IT software and CLiq software; seismically-induced settlement analysis; lateral spreading analysis; slope stability analysis using SLOPE/W software
Geotechnical Design Parameters	Bearing capacities; shear strength parameters	Shear strength parameters	Modulus of subgrade reaction; sheet pile design
Depth to Groundwater	Unknown, not reported	Unknown, not reported	10 ft to 11 ft bgs
Soil Corrosivity	Unknown, not reported	Not Applicable or No Data	- Severely corrosive to metals - Severely corrosive to concrete
Liquefaction Potential	- Based on data from Report No. 16 herein, soils approx. 0 ft to 40 ft bgs are potentially liquefiable, locally interbedded with denser/stiffer soils	Not Applicable or No Data	- Report states soils approx. 15 ft to 20 ft bgs are potentially liquefiable
Peat Layers	Unknown, not distinguished on logs and/or boring logs incomplete	Not Applicable or No Data	Not encountered
Soft Clay / Silt Layers	Unknown, not distinguished on logs and/or boring logs incomplete	Not Applicable or No Data	Not encountered
Additional Comments	Not Applicable or No Data	- Slope stability static factors of safety ≥ 1.5	- Newport-Inglewood Fault Zone is surface rupture hazard - Seismically-induced settlement generally 1.4 inch to 3.6 inches (CPT-3 shows 8.6 inches), differential settlement up to 1.2 inches/100 ft - Liquefiable soils susceptible to lateral spreading, with displacements generally 1.6 ft to 3.6 ft (CPT-3 shows up to 9 ft) - Slope stability static and pseudo-static factors of safety ≥ 1.3 (but less than 1 for post-liquefaction)

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR
WESTMINSTER CHANNELS PROJECT

Report No.	10	11	12
Subreach No.	1.4, 1.5	1.4	19.1
Title	Geotechnical Investigation, East Garden Grove - Wintersburg Channel (C05) Levee Soil Mix Project, Groundwater Impact Evaluation, Station 37+00 to Station 102+00, Huntington Beach, California, 283 pp.	Final Geotechnical Investigation Report, East Garden Grove - Wintersburg Channel Improvements from Graham Street to Warner Avenue (Station 75+00 to 100+00), Huntington Beach, Orange County, California, 343 pp.	Deep Soil Mix Column Levee Structure, East Garden Grove - Wintersburg Channel Improvement, Huntington Ceach, Orange County, California, 29 pp.
Author	Hushmand Associates, Inc.	Earth Mechanics, Inc.	Earth Mechanics, Inc.
Date	5/20/2010	10/15/2009	5/5/2008
Field Exploration	- 10 borings (hollow-stem auger), up to 20 ft deep - 23 CPTs, up to 30 ft deep - Installed 33 groundwater monitoring wells	- 4 borings (rotary wash), up to 80 ft deep - 26 CPTs, up to 80 ft deep	Not Applicable or No Data
Field / Laboratory Tests	Blow Counts; Pore Presssure Dissipation Test; Moisture Content; Unit Weight; Sieve; Atterberg Limits; Triaxial Permeability	Blow Counts; Pore Presssure Dissipation Test; Moisture Content; Unit Weight; Sieve; #200 Wash; Sand Equivalent; Atterberg Limits; Max. Dry Density/Optimum Moisture; Corrosivity; Direct Shear; Consolidation; Unconsolidated Undrained Triaxial	Not Applicable or No Data
Maps	Map with exploration locations; groundwater contours for various time periods	Simplified plan sheets with exploration locations	Not Applicable or No Data
Sections / Profiles	- Boring logs; CPT logs; 4 schematic / interpreted cross sections	Boring logs are missing from report (appears to be clerical error); CPT logs; 5 schematic / interpreted cross sections	Schematic Deep Soil Mix column diagrams
Geotechnical Analyses	Groundwater/hydrogeological impacts and mitigation measures of proposed soil mixing	Liquefaction analysis using CLiq software; seismically-induced settlement analysis using CLiq software	Slope stability analysis using SLIDE software; lateral deformation analysis of levee with Deep Soil Mix columns
Geotechnical Design Parameters	Not Applicable or No Data	Modulus of subgrade reaction	Not Applicable or No Data
Depth to Groundwater	5 ft to 12 ft bgs	12 ft to 15 ft bgs (one location 24 ft bgs)	Not Applicable or No Data
Soil Corrosivity	Not Applicable or No Data	- Moderately to highly corrosive to metals - Non-corrosive to concrete	Not Applicable or No Data
Liquefaction Potential	- Report does not comment on liquefaction - Boring logs show low blow count cohesionless soils approx. 0 ft to 25 ft bgs, locally interbedded with denser/stiffer soils	- Report states soils approx. 10 ft to 60 ft bgs are potentially liquefiable	Not detailed in report, but Deep Soil Mix columns are planned to mitigate liquefaction hazard
Peat Layers	Not encountered	- Top of peat locally 30 ft to 35 ft bgs, thickness varies approx. 1 ft to 5 ft - Top of soft organic clay varies significantly between 23 ft to 36 ft bgs, thickness varies from approx. 5 ft to 10 ft	Not Applicable or No Data
Soft Clay / Silt Layers	Top of soft clay/silt and/or soft silt generally at ground surface, thickness approx. 5 to 6 ft; locally 5-foot thick layer near 12 ft bgs	Generally, interbedded soft clay/silt between ground surface and 15 ft bgs, thickness varies significantly between 5 ft and 25 ft	Not Applicable or No Data
Additional Comments	- Stratigraphy is generally saturated sand layer underlain by low permeability silt/clay layer; therefore perched groundwater conditions, and groundwater elevation varies with rainfall	- Seismically-induced settlement generally 0.9 inch to 3.75 inches, average 2.3 inches (CPT-08-13 shows 5.7 inches)	- Lateral deformations range from 2 inches to 12 inches - Additional Deep Soil Mix specifications for construction QA/QC should be developed - Locally, potentially liquefiable layers extended deeper than DSM is proposed - Recommended additional explorations/borings - Lateral earth pressures recommended by AES (6/29/2005; Report No. 15 herein) are adequate, but passive pressure could be re-evaluated based on groundwater Elev. 0 ft

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	13	14	15
Subreach No.	1.6	1.5	1.5
Title	Geotechnical Engineering Investigation, East Garden Grove - Wintersburg Channel (C05) Emergency Project, North Levee, Station 36+00 to Station 50+00, Huntington Beach, Orange County, California, 170 pp.	Geotechnical Review and Feasibility Evaluation, Proposed Levee Improvements, East Garden Grove - Wintersburg Channel Station 48+00 to 74+25 (C05), 92 pp.	Final Report of Geotechnical Investigation, Proposed East Garden Grove Wintersburg Channel C05 Improvement, Southwest of Graham Street, Southern Levee, from Station No. 48+00 to Station No. 74+25, Huntington Beach, California, 275 pp.
Author	Hushmand Associates, Inc.	Advanced Earth Science, Inc.	MACTEC Engineering and Consulting, Inc.
Date	12/2007	6/29/2005	12/2/2004
Field Exploration	- 5 borings (hollow-stem auger), up to 70 ft deep	- None; review of MACTEC 12/2/2004 reports (Report Nos. 15, 19 & 20 herein), which contained 7 borings and 33 CPTs up to 70 ft deep	- 5 borings (rotary wash), up to 70 ft deep - 33 CPTs, up to 70 ft deep - Also 2 borings from MACTEC 2003 study (Report Nos. 19 & 20 herein) and 10 borings from LeRoy Crandall & Associates 1988 study
Field / Laboratory Tests	Blow Counts; Moisture Content; Unit Weight; Sieve; Sand Equivalent; Atterberg Limits; Corrosivity; Direct Shear; Consolidation	- None; utilized data from MACTEC 12/2/2004 reports (Report Nos. 15, 19 & 20 herein)	Blow Counts; Pore Pressure Dissipation Test; Moisture Content; Unit Weight; Sieve; Hydrometer; Atterberg Limits; Hydraulic Conductivity; Corrosivity; Direct Shear; Consolidation
Maps	Map with exploration locations	Not Applicable or No Data	Map with exploration locations
Sections / Profiles	Boring logs; 1 schematic / interpreted cross section	Not Applicable or No Data	- Boring logs; CPT logs; 6 schematic / interpreted cross sections
Geotechnical Analyses	Liquefaction analysis; seismically-induced settlement analysis; slope stability analysis using GSTABL7 software	Liquefaction analysis; seismically-induced settlement analysis; geotechnical feasibility of five proposed alternative levee improvements	- Liquefaction analysis; seismically-induced settlement analysis; seepage analysis using SEEP/W software; scour analysis
Geotechnical Design Parameters	- Modulus of subgrade reaction; bearing capacity; sheet pile design	- Bearing capacity; shear strength parameters	- Lateral earth pressures; modulus of subgrade reaction; bearing capacity; soil permeability; cut-off wall
Depth to Groundwater	10 ft to 13 ft bgs	Based on analysis of data from MACTEC 12/2/2004 reports, 11 ft to 22 ft bgs (AES preferred range is 11 ft to 13 ft bgs)	12 ft to 17 ft bgs
Soil Corrosivity	- Severely corrosive to metals - Severely corrosive to concrete	Not Applicable or No Data	- Severely corrosive to metals - Non-corrosive to concrete
Liquefaction Potential	- Report states site soils are highly liquefiable - Boring logs show low blow count cohesionless soils approx. 10 ft to 40 ft bgs, locally interbedded with denser/stiffer soils	- Based on data from MACTEC 12/2/2004 reports, soils in upper 50 ft to 60 ft bgs are potentially liquefiable	- Report states soils approx. 20 ft to 30 ft bgs are highly liquefiable and soils approx. 50 ft to 60 ft bgs are potentially liquefiable
Peat Layers	- Top of peat locally 47 ft bgs; thickness approx. 2 ft to 5 ft - Soft organic clay not encountered	Not Applicable or No Data	- Top of peat locally 34 ft to 49 ft bgs, thickness approx. 1 ft to 2 ft - Soft organic clay/silt not encountered
Soft Clay / Silt Layers	Top of soft clay and/or silt generally 37 ft to 40 ft bgs, thickness approx. 5 ft to 6 ft, generally interbedded with layers of loose to dense sand, silt, or sand with silt; locally top of soft clay and/or silt near 6 ft bgs, 2 ft to 3 ft thick	Based on data from MACTEC 12/2/2004 reports, undifferentiated soft soils between ground surface and approx. 40 ft bgs with interbedded stiffer/denser layers	Top of soft clay and/or silt varies significantly 7 ft to 20 ft bgs, thickness varies significantly from approx. 4 ft to 43 ft, typically interbedded with layers of loose to dense sand, silt, and/or sand with silt
Additional Comments	- First water is perched aquifer - Near-surface soils potentially expansive - Newport-Inglewood Fault Zone is surface-rupture hazard - Liquefaction and lateral spreading hazards are high - Seismically-induced settlement up to 12 inches - Without sheet piles: slope stability static factor of safety > 1.8; pseudo-static factor of safety = 1.16; several feet of later deformation in design earthquake - Locally, materials beneath piles could liquefy	- Seismically-induced settlement generally 2 inches to 8 inches, differential settlement up to 3.5 inches/80 ft - Settlement of soil due to structural loading generally 1 inch, differential settlement generally 0.5 inch/80 ft - Liquefiable soils susceptible to lateral spreading, with displacements generally 2 ft to 3 ft - Seismic/liquefaction loading will produce bearing failure in proposed RCB; proposed RCB also subject to buoyancy - Recommended evaluating other liquefaction mitigation measures and considering acceptable damage criteria/risk levels	- Seismically-induced settlement generally 3 inches to 10 inches, differential settlement up to 4 inches/80 ft - Liquefiable soils susceptible to lateral spreading, with displacements generally 2 ft to 3 ft - Seismic/liquefaction loading will produce bearing failure in proposed RCB

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	16	17	18
Subreach No.	1.1, 1.2, 1.3, 1.4 1.5	1.4, 1.5	1.4, 1.5
Title	Geotechnical Investigation Materials Report, East Garden Grove Wintersburg Channel C05 Station 50+00 to Station 152+00, 131 pp.	Review of Water Quality Enhancement and Perched Water Buffer Components for East Garden Grove - Wintersburg Channel (C05) Levee Improvements, 9 pp.	East Garden Grove - Wintersburg Channel OCFCD Facility #C05, Quantitative Engineering Analysis of North Levee Downstream of Graham Street, 31 pp.
Author	Orange County Public Facilities & Resources Dept.	WRC Consulting Services, Inc.	Orange County Resources and Development Management Dept.
Date	3/1/2001	5/23/2012	9/25/2007
Field Exploration	9 borings (hollow-stem auger), up to 40 ft deep	Not Applicable or No Data	Not Applicable or No Data
Field / Laboratory Tests	Blow Counts; Moisture Content; Sieve; San Equivalent; Atterberg Limits; Organic Content; Corrosivity; Direct Shear; Consolidation	Not Applicable or No Data	None; utilized data from MACTEC 5/23/2003 report
Maps	Map with exploration locations	Not Applicable or No Data	Not Applicable or No Data
Sections / Profiles	Boring logs	Not Applicable or No Data	Not Applicable or No Data
Geotechnical Analyses	Consolidation settlement; slope stability analysis	Not Applicable or No Data	Slope stability analysis
Geotechnical Design Parameters	- Lateral earth pressures; bearing capacity; sheet pile design	Not Applicable or No Data	Shear strength
Depth to Groundwater	12 ft to 21 ft bgs	Generally 3 ft bgs	Not Applicable or No Data
Soil Corrosivity	- Severely corrosive to metals - Severely corrosive to concrete	Not Applicable or No Data	Not Applicable or No Data
Liquefaction Potential	- Report states site soils are liquefiable - Boring logs show low blow count cohesionless soils approx. 0 ft to 40 ft bgs, locally interbedded with denser/stiffer soils	Not Applicable or No Data	Not Applicable or No Data
Peat Layers	- Uncertain, not distinguished on logs - Top of peat or soft organics locally may be 19 ft to 35 ft bgs, thickness unknown but may be ~2 ft	Not Applicable or No Data	Not Applicable or No Data
Soft Clay / Silt Layers	Top of soft clay and/or silt locally may be near 29 ft bgs; thickness unknown but may be ~1 ft	Not Applicable or No Data	Not Applicable or No Data
Additional Comments	- Potentially liquefiable soils also potentially subject to seismically-induced settlement, and potentially subject to lateral spreading - Settlement of soil due to structural loading at 25 ft bgs up to 1.6 ft in 133 days expected for a 15 ft high concrete wall - Newport-Inglewood Fault Zone is surface-rupture hazard - Slope stability static > 3	- Local hydrogeology is complicated by shallow (< 50 ft bgs) pervious and impervious zones, so seepage analysis for sheet piles is important - Recommended developing groundwater monitoring program for before and during sheet pile construction - Subsidence near Slater Pump Station in January 2011 produced multiple sinkholes up to 3 ft by 2 ft wide and 3 ft deep - Potential for significant subsidence during pumping due to shallow saturated granular soils	- North levee slopes in study area are unstable in 2007 configuration (due to 2005 storm damage)

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR
WESTMINSTER CHANNELS PROJECT

Report No.	19 & 20	28	29
Subreach No.	1.4, 1.5	1.5	N/A
Title	Report of Geotechnical Investigation, Proposed East Garden Grove Wintersburg Channel C05 Improvement Southwest of Graham Street, Northern Levee From Station No. 48+00 to Station No. 74+25, Huntington Beach, California, 99 pp.	Preliminary Geotechnical Investigation, Proposed Residential Development, Tentative Tract 15377, City of Huntington Beach, California, and Tentative Tract 15419, County of Orange, California, 264 pp.	Westminster Feasibility Study Preliminary Draft Baseline Conditions Report, 161 pp.
Author	MACTEC Engineering and Consulting, Inc.	Pacific Soils Engineering, Inc.	USACOE
Date	12/2/2004	2/2/1998	5/2007
Field Exploration	- 2 borings (rotary wash), up to 70 ft deep - Also 10 borings from LeRoy Crandall & Associates 1988 study and 2 borings from Orange County study (Report No. 18 herein)	- 8 borings (hollow-stem auger), up to 50 ft deep - 65 CPTs, up to 60 ft deep - 12 test pits, up to 15 ft deep - Installed 4 groundwater monitoring wells - Also reviewed several dozen explorations (borings, CPTs, and trenches) by other consultants	Not Applicable or No Data
Field / Laboratory Tests	Blow Counts; Moisture Content; Unit Weight; Sieve; Hydrometer; Atterberg Limits; Hydraulic Conductivity; Corrosivity; Direct Shear; Consolidation	Blow Counts; Pore Pressure Dissipation Test; Moisture Content; Unit Weight; Sieve; Atterberg Limits; Expansion Index; Max. Dry Density/Optimum Moisture; Corrosivity; Direct Shear; Consolidation	Not Applicable or No Data
Maps	Map with exploration locations	None	Groundwater contours for Orange County
Sections / Profiles	Boring logs (one page missing from report; appears to be clerical error); 1 schematic / interpreted cross sections, illegible copy	Boring logs; CPT logs; Trench logs	Not Applicable or No Data
Geotechnical Analyses	- Liquefaction analysis; seismically-induced settlement analysis; seepage analysis using SEEP2D software; scour analysis	- Liquefaction analysis using CPTINT software; seismically-induced settlement analysis	Not Applicable or No Data
Geotechnical Design Parameters	Sheet pile design; pile capacities	None	Not Applicable or No Data
Depth to Groundwater	13 ft to 17 ft bgs	3 ft to 20 ft bgs	Not Applicable or No Data
Soil Corrosivity	- Severely corrosive to metals - Moderately corrosive to concrete	- Severely corrosive to metals - Moderately to severely corrosive to concrete	Not Applicable or No Data
Liquefaction Potential	- Report states site soils are liquefiable - Boring logs show low blow count cohesionless soils approx. 0 ft to 50 ft bgs, locally interbedded with denser/stiffer soils	- Report states sandy soils below 10 ft bgs are moderately liquefiable - Boring logs show low blow count cohesionless soils approx. 0 ft to 50 ft bgs, locally interbedded with denser/stiffer soils	Not Applicable or No Data
Peat Layers	- Top of peat locally 48 ft bgs, thickness approx. 1 ft to 2 ft - Soft organic clay/silt not encountered	- Top of peat varies significantly: locally 3 ft to 4 ft bgs, thickness approx. 1 ft to 2 ft; also locally 26 ft bgs, thickness approx. 1 ft - Soft organic clay/silt not encountered	Not Applicable or No Data
Soft Clay / Silt Layers	Top of soft clay and/or silt varies significantly between 27 ft to 50 ft bgs, thickness varies significantly from approx. 2 ft to 18 ft, interbedded with layers of loose to dense sand, silt, and/or sand with silt	Top of soft clay and/or silt varies significantly: often between ground surface and 5 ft bgs, thickness varies from approx. 1 ft to 3 ft; locally 14 ft to 25 ft bgs, thickness varies approx. 2 ft to 7 ft	Not Applicable or No Data
Additional Comments	- Seismically-induced settlement up to 4 inches, differential settlement up to 3 inches - Site is not suitable for proposed RCB modification due to liquefiable soils - Recommended additional investigation in the form of CPT	- Excerpt from Report No. 15 herein	- Settlement of soil due to structural loading of 0.5 inch to 1 inch per foot of fill can be expected (more when peat is at the surface) - In situ soils have low expansion potential

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	30	32	33
Subreach No.	1.4, 1.5	20.1, 20.2, 21.1, 21.2, 21.3, 21.4 22.1, 22.2, 22.3, 22.4, 22.5, 22.6, 22.7, 22.8, 22.9, 22.10	All of C05 and C06
Title	Report Synopsis for Westminster, East Garden Grove Flood Risk Management Study, 40 pp.	Project Report for Westminster Channel (C04), Bolsa Chica Confluence (C02) to Garden Grove Freeway (SR-22), 237 pp.	Project Report for East Garden Grove - Wintersburg (C05) and Oceanview (C06) Channels, 279 pp.
Author	unknown	WRC Consulting Services, Inc.	Williamson & Schmid
Date	2/2014	2/1/2005	12/1/1994
Field Exploration	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Field / Laboratory Tests	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Maps	Not Applicable or No Data	Civil design plan and profile sheets of proposed improvements	Not Applicable or No Data
Sections / Profiles	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Geotechnical Analyses	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Geotechnical Design Parameters	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Depth to Groundwater	Not Applicable or No Data	Not Applicable or No Data	Generally 0 ft to 10 ft bgs
Soil Corrosivity	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Liquefaction Potential	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Peat Layers	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Soft Clay / Silt Layers	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Additional Comments	Not Applicable or No Data	- Identification and evaluation of numerous alternatives by reach with recommendations - Photographs of upstream and downstream views of most crossings (dates uncertain, but appear to be ca. 2000)	- Newport-Inglewood Fault Zone is surface-rupture hazard - Settlement of soil due to structural loading could be 3 inches to 24 inches, differential settlements could exceed 12 inches/50 ft - Potential for settlement due to dewatering - Soils are moderately to highly compressible saturated clays, silts, and peat with approx. thickness of 5 ft to 30 ft; therefore, soils are subject to consolidation during construction and dewatering - Recommended concrete structures in areas of organics should be supported on piles that extend below organic deposits

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR
WESTMINSTER CHANNELS PROJECT

Report No.	34	35	36
Subreach No.	2.1, 20.9	20.9	20.3, 20.4
Title	As-Built Plans for the Construction of Bolsa Chica Channel, Tidelands to Cerritos Avenue, 32 pp.	As-Built Plans for Construction of Bolsa Chica Channel (Facility No. C02) from Westminster Channel (Sta. 86+00) to Anaheim-Barber City Channel (Sta. 166+00), 8 pp.	Plans for the Construction of Westminster Channel, McFadden Avenue to Sta. 92+20 and at Graham Street, 10 pp.
Author	Orange County Flood Control District	Orange County Environmental Management Agency	Orange County Flood Control District
Date	6/1959	1/1985	12/1962
Field Exploration	- 5 borings (rotary wash), up to 50 ft deep	- 4 borings (drill method unknown), up to 25 ft deep	- 2 borings (drill method unknown), up to 55 ft deep
Field / Laboratory Tests	Blow Counts; Moisture Content; Unit Weight	Moisture Content; Unit Weight; Sand Equivalent; Corrosivity; Direct Shear	Blow Counts; Moisture Content; Unit Weight
Maps	Map with exploration locations	Map with exploration locations	None
Sections / Profiles	Boring logs	Boring logs	Boring logs
Geotechnical Analyses	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Geotechnical Design Parameters	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Depth to Groundwater	6 ft to 13 ft bgs	12 ft to 16 ft bgs	Unknown, not distinguished on logs
Soil Corrosivity	Not Applicable or No Data	- Negligibly to moderately corrosive to concrete	Not Applicable or No Data
Liquefaction Potential	- Boring logs show low blow count cohesionless soils approx. 10 ft to 30 ft bgs, locally interbedded with denser/stiffer soils	Not Applicable or No Data	Boring logs show low blow count cohesionless soils approx. 0 ft to 20 ft bgs
Peat Layers	Not encountered	Not Applicable or No Data	Not encountered
Soft Clay / Silt Layers	Top of soft clay/silt 8 ft to 15 ft bgs, thickness varies significantly from approx. 2 ft to 14 ft	Not Applicable or No Data	Not encountered
Additional Comments	- No report available	- No report available	- No report available

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	37	38	39
Subreach No.	20.1	4.1, 4.2, 4.3	3.3, 3.4, 13.3
Title	Plans for Construction of Westminster Channel, Facility No. C04, from D/S Goldenwest Street to U/S San Diego Fwy. (I-405), 11 pp.	Plans for Construction of Westminster Channel, Facility No. C04, from U/S of Magnolia to D/S of Brookhurst, 12 pp.	As-Built Plans for the Construction of East Garden Grove–Wintersburg Channel, Tidelands to Huntington Beach Blvd., 30 pp.
Author	Orange County Environmental Management Agency	Orange County Environmental Management Agency	Orange County Flood Control District
Date	2/1991	3/1992	9/1959
Field Exploration	- 3 borings (drill method unknown), up to 15 ft deep	- 35 borings (drill method unknown), up to 15 ft deep	- 20 borings (hollow-stem auger and rotary wash), up to 60 ft deep
Field / Laboratory Tests	Blow Counts; Moisture Content; Unit Weight; Sand Equivalent; Corrosivity; Direct Shear	Moisture Content; Unit Weight; Sand Equivalent; Corrosivity; Direct Shear	Blow Counts; Moisture Content; Unit Weight
Maps	Map with exploration locations	Map with exploration locations	Map with exploration locations
Sections / Profiles	Boring logs	Boring logs	Boring logs
Geotechnical Analyses	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Geotechnical Design Parameters	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Depth to Groundwater	10 ft to 12 ft bgs	11 ft bgs	5 ft to 18 ft bgs
Soil Corrosivity	- Non-corrosive to concrete	- Non-corrosive to concrete	Not Applicable or No Data
Liquefaction Potential	Boring logs show low blow count cohesionless soils approx. 0 ft to 15 ft bgs	Not Applicable or No Data	- Boring logs show low blow count cohesionless soils approx. 0 ft to 50 ft bgs, locally interbedded with denser/stiffer soils, highly variable
Peat Layers	Not Applicable or No Data	Not Applicable or No Data	- Top of peat varies between 3 ft to 7 ft bgs, thickness generally approx. 1 ft to 3 ft, locally 13 ft thick; locally, top of peat 17 ft bgs, thickness approx. 10 ft - Soft organic clay/silt not encountered
Soft Clay / Silt Layers	Not Applicable or No Data	Not Applicable or No Data	Top of soft clay and/or silt varies significantly from ground surface to 27 ft bgs, thickness varies significantly from approx. 3 ft to 25 ft, locally 40 ft to 44 ft
Additional Comments	- No report available	- No report available	- No report available

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	40	41	42
Subreach No.	3.1, 3.2, 4.1, 4.2, 4.3, 4.4, 4.5, 5.1, 5.2, 5.3, 5.4, 5.5, 5.6, 6.1, 6.2, 7.1	8.1, 8.2, 8.3, 8.4, 9.1, 9.2, 9.3, 9.4, 9.5, 10.1, 10.2	1.4
Title	Plans for the Construction of East Garden Grove–Wintersburg Channel, Beach Blvd. to Newhope Street, 25 pp.	Plans for the Construction of East Garden Grove-Wintersburg Channel, Newhope St. to Haster Retarding Basin, 22 pp.	Plans for the Southern Levee Restoration of East Garden Grove Wintersburg Channel, Facility No. C05, from Graham St. to Warner Ave., 13 pp.
Author	Orange County Flood Control District	Orange County Flood Control District	Orange County Environmental Management Agency
Date	4/1960	6/1961	9/1993
Field Exploration	- 10 borings (rotary wash), up to 52 ft deep	- 10 borings (rotary wash), up to 25 ft deep	- 3 borings (rotary wash), up to 26 ft deep
Field / Laboratory Tests	Blow Counts; Moisture Content; Unit Weight	Blow Counts; Moisture Content; Unit Weight	Blow Counts; Moisture Content; Unit Weight; Sand Equivalent; Corrosivity; Direct Shear
Maps	Map with exploration locations	Map with exploration locations	Map with exploration locations
Sections / Profiles	Boring logs	Boring logs	Boring logs
Geotechnical Analyses	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Geotechnical Design Parameters	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Depth to Groundwater	6 ft to 13 ft bgs, locally not encountered	6 ft to 12 ft bgs, locally not encountered	11 ft bgs
Soil Corrosivity	Not Applicable or No Data	Not Applicable or No Data	Generally non-corrosive to concrete (one location moderately corrosive to concrete)
Liquefaction Potential	- Boring logs show low blow count cohesionless soils approx. 0 ft to 55 ft bgs, locally interbedded with denser/stiffer soils, highly variable	- Boring logs show low blow count cohesionless soils approx. 0 ft to 50 ft bgs, locally interbedded with denser/stiffer soils	- Boring logs show low blow count cohesionless soils approx. 0 ft to 10 ft bgs
Peat Layers	- Top of peat varies significantly between ground surface and 12 ft bgs, locally interbedded with sand and/or silty sand package approx. 25 ft thick with peat layers up to 6 ft thick - Soft organic clay/silt not encountered	Not encountered	Not Applicable or No Data
Soft Clay / Silt Layers	Top of soft clay and/or silt varies significantly between ground surface and 23 ft bgs, thickness varies from approx. 1 ft to 10 ft	Top of soft clay/silt generally 13 ft to 17 ft bgs, thickness varies from approx. 1 ft to 5 ft; locally top of soft clay/silt 2 ft to 6 ft bgs, thickness varies from approx. 2 ft to 10 ft	Not Applicable or No Data
Additional Comments	- No report available	- No report available	- No report available

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	43	44	45
Subreach No.	2.2, 2.3	2.2, 3.4, 13.3	3.1, 4.5
Title	Plans for the Construction of East Garden Grove–Wintersburg Channel O.C.F.C.D. Facility No. C05, from 411 m D/S of Golden West St. to 349 m U/S of Golden West St. [sic], 33 pp.	Plans for the Construction of East Garden Grove–Wintersburg Channel O.C.F.C.D. Facility No. C05, and Ocean View Channel O.C.F.C.D. Facility No. C06, from 349 m D/S of Golden West St. [sic] to 350 m U/S of Gothard St., 37 pp.	Plans for Improvement of Newland Storm Channel, C05 Confluence to D/S Whitley Ave (Facility No. C05S01), Station 00+34.11 to Station 61+40, 79 pp.
Author	Orange County Environmental Management Agency	Orange County Environmental Management Agency	County of Orange Public Works
Date	4/1998	2/2001	1/2014
Field Exploration	- 14 borings (hollow-stem auger), up to 51 ft deep	- 10 borings (hollow-stem auger), up to 51 ft deep	- 20 borings (rotary wash), up to 50 ft deep
Field / Laboratory Tests	Blow Counts; Moisture Content; Unit Weight	Blow Counts; Moisture Content; Unit Weight	- Blow Counts; Moisture Content; Unit Weight; Unconfined Compression - Other tests reportedly completed but results not shown
Maps	Map with exploration locations	Map with exploration locations	Map with exploration locations
Sections / Profiles	Boring logs	Boring logs	Boring logs
Geotechnical Analyses	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Geotechnical Design Parameters	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Depth to Groundwater	12 ft to 18 ft bgs	18 ft to 27 ft bgs	5 ft to 17 ft bgs
Soil Corrosivity	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Liquefaction Potential	- Boring logs show low blow count cohesionless soils approx. 4 ft to 45 ft bgs, locally interbedded with denser/stiffer soils	- Boring logs show low blow count cohesionless soils approx. 4 ft to 40 ft bgs, locally interbedded with denser/stiffer soils	- Boring logs show low blow count cohesionless soils approx. 0 ft to 40 ft bgs, locally interbedded with denser/stiffer soils
Peat Layers	- Top of peat generally 7 ft to 15 ft bgs, thickness approx. 0.5 ft to 6 ft - Soft organic clay/silt not encountered	- Typically peat not encountered; locally top of 17 ft bgs, thickness approx. 4 ft to 5 ft - Soft organic clay/silt not encountered	- Peat not encountered - Top of soft organic clay/silt generally 10 ft to 25 ft bgs, thickness approx. 3 ft to 8 ft
Soft Clay / Silt Layers	Top of soft clay and/or silt varies significantly between 7 ft and 24 ft bgs, thickness varies 1 ft to 6 ft; locally top of soft clay and/or silt 35 ft to 39 ft bgs, 1 ft to 5 ft thick	Top of soft clay and/or silt varies significantly between 5 ft to 22 ft bgs, thickness genearlly from approx. 3 ft to 6 ft; often interbedded with denser/stiffer soils	Top of soft clay and/or silt varies significantly between 5 ft to 30 ft bgs, thickness varies from approx. 1 ft to 15 ft, often interbedded with denser/stiffer soils
Additional Comments	- No report available	- No report available	- No report available

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR
WESTMINSTER CHANNELS PROJECT

Report No.	46	47	48
Subreach No.	16.1	20.9	18.1
Title	Plans for the Construction of That Portion of Ocean View Channel, Facility C06, Magnolia Street to Bushard Street, 10 pp.	Geotechnical Services, Soil Sampling and Laboratory Testing, Huntington Harbour, California, 20 pp.	Liquefaction Potential, Sycamore Valley Apartment Complex, 10349 Slater Avenue, Fountain Valley, California, 35 pp.
Author	Orange County Environmental Management Agency	Diaz • Yourman & Associates	Diaz • Yourman & Associates
Date	5/1983	6/21/1994	2/8/1996
Field Exploration	- 3 borings (24 inch diameter bucket auger), up to 20 ft deep	- 6 shallow hand-driven sample holes, depth unknown	- 3 CPTs, up to 60 ft deep
Field / Laboratory Tests	- Blow Counts; Moisture Content; Unit Weight - Other tests reportedly completed but results not shown	- Pocket Penetrometer; Hand Vane Shear; Moisture Content; Unit Weight; Sieve; Atterberg Limits; Direct Shear	- Pore Pressure Dissipation Test
Maps	Map with exploration locations	Map with exploration locations	Map with exploration locations
Sections / Profiles	Boring logs	Not Applicable or No Data	Not Applicable or No Data
Geotechnical Analyses	Not Applicable or No Data	Not Applicable or No Data	Liquefaction analysis; seismically-induced settlement analysis
Geotechnical Design Parameters	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Depth to Groundwater	19 ft bgs, locally not encountered	Not Applicable or No Data (within Huntington Harbor)	Not Applicable or No Data
Soil Corrosivity	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Liquefaction Potential	- Boring logs show low blow count cohesionless soils approx. 3 ft to 9 ft bgs	Not Applicable or No Data	- Loose sand or silty sandy not encountered; however, the soils are still potentially liquefiable at MCE-level
Peat Layers	- Top of peat locally between 4 ft and 10 ft bgs, thickness approx. 1 ft to 2 ft - Soft organic clay/silt not encountered	Not Applicable or No Data	Not Applicable or No Data
Soft Clay / Silt Layers	Top of soft clay and/or silt varies significantly between 3 ft and 20 ft bgs, thickness varies significantly from approx. 1 ft to 17 ft, typically interbedded with slightly denser/stiffer soils	Top of soft clay and/or silt at mudline; thickness unknown	Not Applicable or No Data
Additional Comments	- No report available	Not Applicable or No Data	- Seismically-induced settlement up to 2 inches, negligible differential settlement

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	49	50	51
Subreach No.	21.4	3.3, 3.4, 14.1, 14.2, 15.1, 20.2, 20.3	20.9
Title	Draft Geotechnical Services for Southern California Edison Fenwick Building Modifications, 14799 Chestnut Street, Westminster, California, 2 pp.	Geotechnical Investigation, Chlorine Containment Equipment Shelters, Wells 6, 7, 9, and 10, Huntington Beach, California, 36 pp.	Initial Site Assessment, Seal Beach Regional Trail, Seal Beach, California, 188 pp.
Author	Diaz • Yourman & Associates	Diaz • Yourman & Associates	Diaz • Yourman & Associates
Date	3/1/2000	9/15/2000	4/5/2004
Field Exploration	- Unknown number of shallow hand-driven sample holes and hand-auger holes, up to 4 ft deep	- 4 borings (hand-auger), up to 4.5 ft deep	Not Applicable or No Data
Field / Laboratory Tests	Direct Shear	Pocket Penetrometer; Moisture Content; Unit Weight; Sieve; Atterberg Limits; Corrosivity	Not Applicable or No Data
Maps	Not Applicable or No Data	Map with exploration locations	Not Applicable or No Data
Sections / Profiles	Not Applicable or No Data	Boring logs	Not Applicable or No Data
Geotechnical Analyses	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Geotechnical Design Parameters	Lateral earth pressures; bearing capacity	Not Applicable or No Data	Not Applicable or No Data
Depth to Groundwater	Not encountered within 4 ft bgs	Not encountered within 4.5 ft bgs	0 ft bgs
Soil Corrosivity	Not Applicable or No Data	- Mildly to severely corrosive to metals - Non-corrosive to concrete	Not Applicable or No Data
Liquefaction Potential	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Peat Layers	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Soft Clay / Silt Layers	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data
Additional Comments	Not Applicable or No Data	Not Applicable or No Data	Not Applicable or No Data

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	52	53	54
Subreach No.	8.1, 9.5	4.3, 22.8	20.2
Title	Geotechnical Investigation, OCTA Annex Building, Garden Grove, California, 72 pp.	Geotechnical Investigation, Moran Street and Bishop Place, Westminster, California, 24 pp.	Geotechnical Investigation, Petroleum Storage Tank, Huntington Beach, California, 71 pp.
Author	Diaz • Yourman & Associates	Diaz • Yourman & Associates	Diaz • Yourman & Associates
Date	5/20/2005	6/1/2007	8/8/2008
Field Exploration	- 4 borings (hand-auger), up to 4.5 ft deep - 4 CPTs, up to 100 ft deep	- 3 borings (hollow-stem auger), up to 32 ft deep	- 2 borings (hollow-stem auger), up to 61 ft deep
Field / Laboratory Tests	Moisture Content; Unit Weight; Sieve; #200 Wash; Atterberg Limits; Max. Dry Density/Optimum Moisture; Corrosivity	Blow Counts; Moisture Content; Unit Weight; Sieve; #200 Wash; Atterberg Limits; Consolidation	Blow Counts; Moisture Content; Unit Weight; Sieve; #200 Wash; Atterberg Limits; Max. Dry Density/Optimum Moisture; Corrosivity; Direct Shear; Consolidation
Maps	Map with exploration locations	None	Map with exploration locations
Sections / Profiles	CPT logs	Boring logs	Boring logs
Geotechnical Analyses	Liquefaction analysis; seismically-induced settlement analysis	Not Applicable or No Data	Slope stability analysis
Geotechnical Design Parameters	Not Applicable or No Data	Not Applicable or No Data	Lateral earth pressures; shear strength
Depth to Groundwater	17 ft bgs	7 ft to 10 ft bgs	40 ft bgs
Soil Corrosivity	- Mildly corrosive to metals - Non-corrosive to concrete	Not Applicable or No Data	- Corrosive to metals - Non-corrosive to concrete
Liquefaction Potential	- Report states site soils are liquefiable - Boring logs show low blow count cohesionless soils approx. 0 ft to 80 ft bgs, locally interbedded with denser/stiffer soils	- Report does not comment on liquefaction - Boring logs show low blow count cohesionless soils approx. 0 ft to 25 ft bgs, locally interbedded with denser/stiffer soils	- Report does not comment on liquefaction - No low blow count cohesionless soils on boring logs
Peat Layers	Not Applicable or No Data	Not Applicable or No Data	Not encountered
Soft Clay / Silt Layers	Not Applicable or No Data	Not Applicable or No Data	Not encountered
Additional Comments	- Seismically-induced settlement up to 7 inches, differential settlement up to 4 inches - Recommended compaction grouting to mitigate liquefaction hazard	- Loose sandy soils in upper 25 ft are potentially liquefiable and typically interbedded with firm clays and/or silts	- Settlement of soil due to structural loading of up to 4 inches - Slope stability static and pseudo-static factors of safety ≥ 1.3

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	55	56	57
Subreach No.	22.10	20.9	2.1, 21.2
Title	Geotechnical Investigation, Street and Drainage Improvements, Westminster, California, 40 pp.	Geotechnical Investigation, Bulkhead Evaluation, Sunset Harbor Maintenance Dredging, Orange County, California, 109 pp.	Geotechnical Investigation, Gothard-Hoover Street Extension, Orange County, California, 87 pp.
Author	Diaz • Yourman & Associates	Diaz • Yourman & Associates	Harding Lawson Associates
Date	6/5/2012	9/27/2013	7/31/1990
Field Exploration	- 5 borings (hollow-stem auger), up to 6 ft deep	- 2 borings (rotary wash), up to 41 ft deep - 6 CPTs, up to 50 ft deep - 2 test pits, up to 5 ft deep	- 9 borings (hollow-stem auger), up to 60 ft deep
Field / Laboratory Tests	Blow Counts; Moisture Content; Unit Weight; #200 Wash; Atterberg Limits; Max. Dry Density/Optimum Moisture; R-value; Corrosivity	Blow Counts; Moisture Content; Unit Weight; #200 Wash; Atterberg Limits; Corrosivity; Direct Shear	Blow Counts; Moisture Content; Unit Weight; Sieve; #200 Wash; Atterberg Limits; Max. Dry Density/Optimum Moisture; R-value; Corrosivity; Unconfined Compression; Direct Shear; Consolidation
Maps	Map with exploration locations	Map with exploration locations	Map with exploration locations
Sections / Profiles	Boring logs	Boring logs; CPT logs	Boring logs
Geotechnical Analyses	Not Applicable or No Data	Liquefaction analysis; seismically-induced settlement analysis; slope stability analysis	Not Applicable or No Data
Geotechnical Design Parameters	Not Applicable or No Data	Lateral earth pressures; shear strength	Lateral earth pressures; bearing capacity; shear strength; pile loading; soil nails
Depth to Groundwater	Not encountered within 6.5 ft bgs	6 ft to 10 ft bgs	- 12 ft to 32 ft bgs - Elev. +18 ft to -2 ft
Soil Corrosivity	- Corrosive to metals - Non-corrosive to concrete	- Severely corrosive to metals - Non-corrosive to concrete	- Corrosive to metals - Non-corrosive to concrete
Liquefaction Potential	- Report does not comment on liquefaction - Boring logs show low blow count cohesionless soils approx. 1 ft to 6 ft bgs	- Report states site soils are liquefiable - Boring logs show low blow count cohesionless soils approx. 0 ft to 23 ft bgs	- Report does not comment on liquefaction - Boring logs show low blow count cohesionless soils approx. 0 ft to 40 ft bgs, locally interbedded with denser/stiffer soils
Peat Layers	Not encountered	Not encountered	- Peat not encountered; however, peat common to local area within 0 ft to 30 ft bgs - Soft organic clay/silt not encountered
Soft Clay / Silt Layers	Top of Soft clay and/or silt locally 3 ft to 6 ft bgs, thickness approx. 1 ft to 2 ft	Top of Soft clay and/or silt locally 4 ft bgs, thickness approx. 8 ft	Soft clay and/or silt locally 1 ft to 5 ft bgs, thickness approx. 2 ft to 3 ft
Additional Comments	Not Applicable or No Data	- Seismically-induced settlement up to 7 inches - Slope stability static factor of safety ≥ 1.2; pseudo-static factor of safety < 1 for deep-seated - Recommended stone columns to mitigate liquefaction hazard	- Report contains existing Bolsa Overhead pile/abutment design parameters - Potential for settlement of loose soils under structural loading - Recommended soil nails due to limited access

TABLE A-1
KEY GEOTECHNICAL DATA REPORTS SUMMARY FOR WESTMINSTER CHANNELS PROJECT

Report No.	58	59
Subreach No.	3.2, 3.3, 3.4	17.1
Title	Geotechnical Investigation, Holiday Inn Hotel, Center Drive, Huntington Beach, California, 92 pp.	Geotechnical Engineering Investigation, Proposed Sam's Club with Fueling Facility and Two Satellite Pads, SWC of Brookhurst Street and Warner Avenue, Fountain Valley, California, 117 pp.
Author	Woodward-Clyde Consultants	Krazen & Associates, Inc.
Date	3/20/1984	3/2/2005
Field Exploration	- 5 borings (hollow-stem auger and rotary wash), up to 100 ft deep - 8 CPTs, up to 60 ft deep	- 56 borings (hollow-stem auger), up to 50 ft deep
Field / Laboratory Tests	Blow Counts; Moisture Content; Unit Weight; Sieve; Atterberg Limits; Organic Content; R-value; Corrosivity; Unconfined Compression; Consolidation	Blow Counts; Moisture Content; Unit Weight; Sieve; R-value; Corrosivity; Unconfined Compression; Direct Shear; Consolidation
Maps	Map with exploration locations	Map with exploration locations
Sections / Profiles	Boring logs; CPT logs	Boring logs
Geotechnical Analyses	Not Applicable or No Data	Liquefaction analysis using Liquefy2 software; seismically-induced settlement analysis
Geotechnical Design Parameters	Lateral earth pressures; pile design	Lateral earth pressures; shear strength
Depth to Groundwater	9 ft to 19 ft bgs	10 ft to 14 ft bgs
Soil Corrosivity	- Moderalte to severely corrosive to metals - Corrosive to concrete	- Corrosivity to metals not reported - Non-corrosive to concrete
Liquefaction Potential	- Report does not comment on liquefaction - No low blow count cohesionless soils on boring logs	- Report states sandy soils below 10 ft bgs are moderately liquefiable - Boring logs show low blow count cohesionless soils approx. 2 ft to 18 ft bgs
Peat Layers	- Top of peat locally 7 ft to 14 ft bgs, thickness approx. 1 ft to 6 ft; may be found as deep as 25 ft or 30 ft bgs - Top of soft organic clay/silt approx. 8 ft to 27 ft bgs, thickness approx. 5 ft	Not encountered
Soft Clay / Silt Layers	Not encountered	Soft clay and/or silt locally approx. 8 ft to 15 ft bgs; thickness varies approx. 4 ft to 12 ft
Additional Comments	- Recommended driven pile foundations	- Seismically-induced settlement up to 3.5 inches, differential settlement up to 2.5 inches - Recommended geogrid, compaction grouting, or deep foundations mitigate liquefaction hazard

Appendix G-2

Geotechnical and Design Considerations for Tunnel Alternative

1 Background

The previous investigations have presented geotechnical data and assessment of foundation conditions necessary for alternatives involving design and construction of near-surface drainage and flood control measures. If alternatives involve tunnels as the primary or supplemental elements of a solution, additional subsurface investigations will be necessary.

Several drainage projects in southern California already in operation or construction rely on tunnels for some or all of their capacity including the Los Angeles Clearwater Project and the Southern California Brine Disposal Tunnel. Additionally there are examples nationwide where tunnels have been incorporated in stormwater runoff management in congested urban areas.

The objectives of the project including potential tunnel elements include:

- Reduce flood hazards along C05/06 and the C02/04 channel systems including risks to life, safety and damages to private and public infrastructure by the year 2035
- Reduce flood impacts in the vicinity of Outer Bolsa Bay and Pacific Coast Hwy by the year 2035

Planning considerations include

- Limit changes to local land use and zoning by limiting channel improvements to those possible within existing rights-of-way
- Minimize impacts to culturally sensitive areas including the Children's Cemetery adjacent to Reach 21 of C04
- Limited change in elevation across watershed limits opportunities for lowering the invert of existing channel systems
- Minimize threat of C05 overflows to Bolsa Chica Ecological Reserve

A conceptual tunnel element would consist of 4 to 6 miles of 24-foot diameter tunnel with 2 or 3 inlet structures as shown in Figure G2-1. The alignment could be optimized using existing right-of-way beneath C-05, thus eliminating additional real estate concerns. The offshore outlet would be far enough out to minimize littoral drift concerns and impacts to the Ecological Reserve – perhaps in the range of ½ mile offshore as shown in Figure G2-2. Based upon existing geological documents a depth of 100-feet or greater would be optimized for efficiency and stability. A concept would be based upon stormwater intake tunnel systems and inlet structures from throughout similar environments for similar purposes as shown in Figure G2-3.

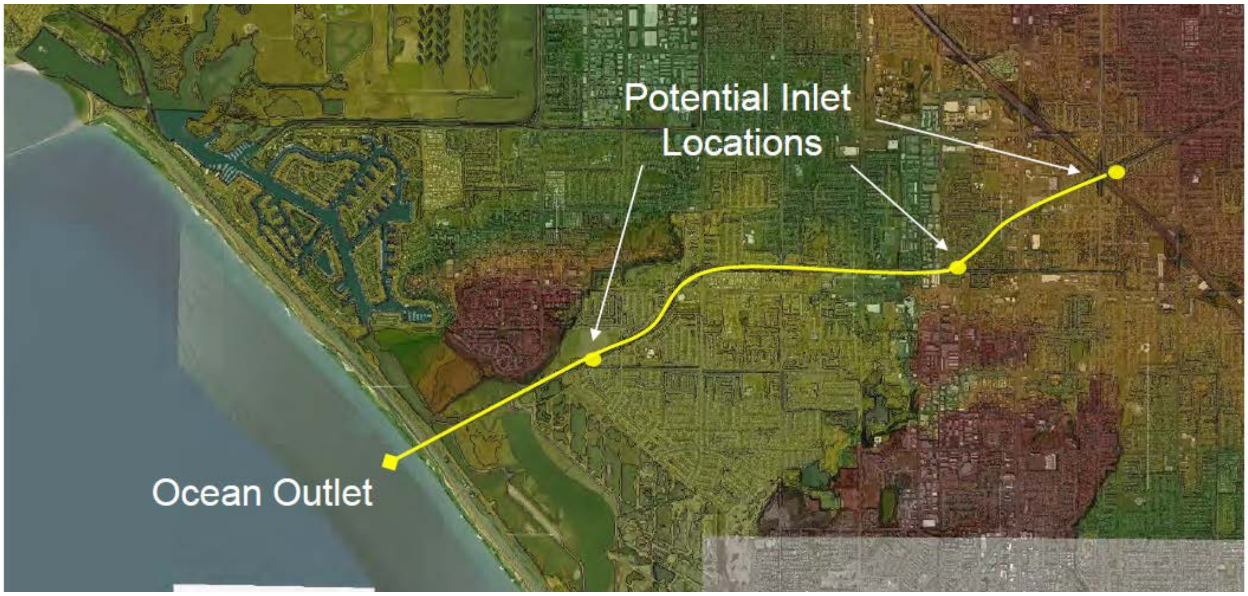


Figure G2-1. Preliminary concept of tunnel alignment and inlet and outlet structures

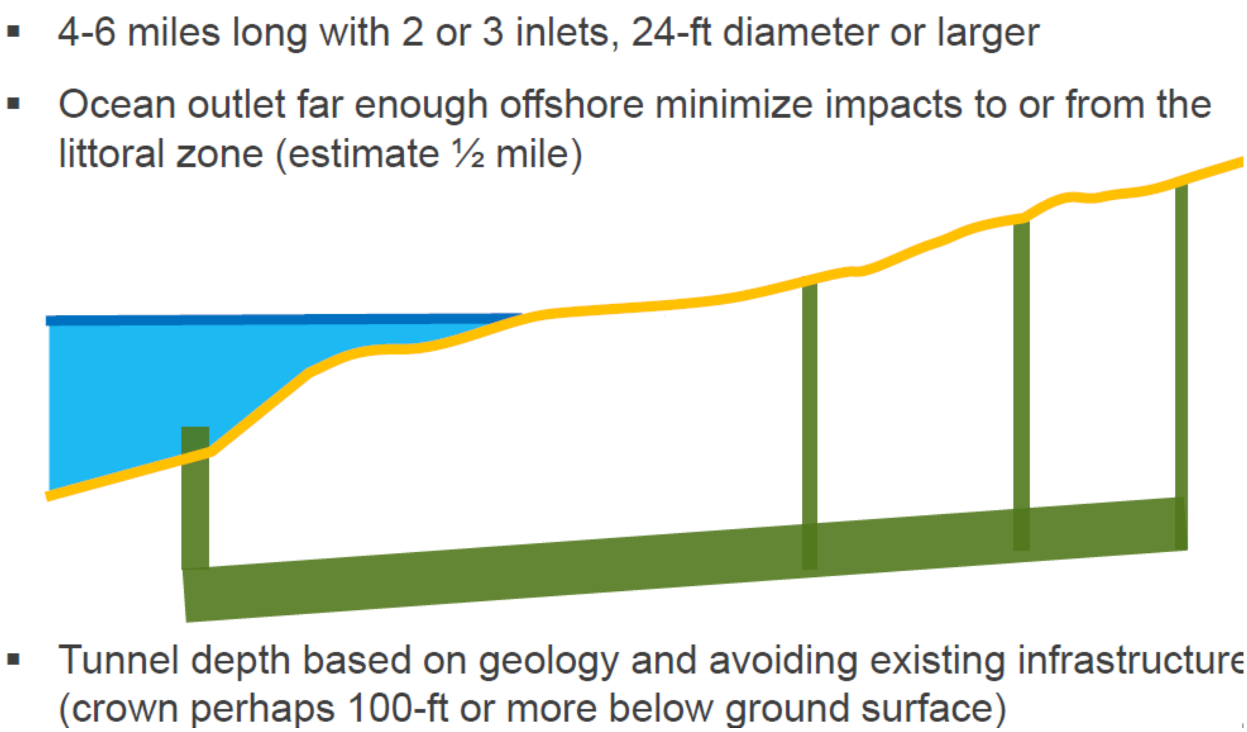


Figure G2-2. Preliminary schematic section of tunnel alignment

Figure G2-3.Typical intake structure with vertical bar trash rack

2 Tunnel System to Address Urban Flooding

2.1 Advantages

Major advantages to tunnel-based solutions to address urban flooding include:

- Tunnels can be constructed rapidly by comparison to surface structures.
- Work can be performed 24 hours/day without impacting traffic, noise or dust
- Fewer surface roads or bridges would require modifications
- Far lower traffic impacts
- Safer and quieter for surrounding community during construction
- Benefits accrued sooner and may offset greater construction costs over lifetime
- Greater flexibility for adding supplemental conduits as needs and funds allow
- Fewer surface structural changes – less real estate impacted
- Avoids Environmentally sensitive or culturally sensitive areas – preserves existing neighborhood character
- Avoids potential near surface contaminated areas /HTRW concerns
- Avoids impacts to near surface infrastructure, pipelines, cables, etc.
- Permits more creative and aesthetic surface features for existing drainage
- Lower operational and maintenance costs

2.2 Disadvantages

Major disadvantages of tunnel based solution to address urban flooding include:

- Greater initial cost
- Requires specialty contractor/equipment
- Does not address bridge/road upgrades that might be included in surface feature alternative design

2.3 Additional Work Required for Tunnel Design

Initial costs and conventional B/C analysis of a tunnel-based alternative may result in this type of solution not being selected, consideration to the non-economic advantages and the difficult to quantify advantages of constructing tunnels. A major surface drainage reconstruction project in a congested residential area will have significant traffic impacts for long periods of time. In contrast to this, tunneling will proceed without significantly impacting the day-to-day happenings in the neighborhoods/communities that will benefit from it.

Geology and geotechnical literature review and investigations necessary for the design and construction of large-diameter (10-30+ feet in diameter) lined tunnels can be a major cost consideration. Core borings into bedrock and a program of geotechnical and geologic testing of both the subsurface materials and the hydrogeologic properties of the subsurface are required to make a sound design. Borings along the tunnel alignments would be recommended on 500-feet centers or closer spacing where geology is more complex or conditions more variable.

A seismic study would be required to address the location(s) where tunnel alignments would cross active seismic features including at least one fault line. The rock/soil mechanical properties will require investigation to refine the design. Testing for soil and rock strength, elasticity, hydraulic conductivity, and potential interaction between groundwater chemistry and lining would be necessary. Investigations would also be necessary to assess subsurface characteristics that might impact worker safety – ground stability, gassy environments, access and egress considerations of the layout, etc.

The tunnel lining in areas of seismic loading would require both a degree of flexibility and redundant hydraulic barrier, however consideration could be given to a free-draining design once construction is completed, as the overall purpose is drainage, rather than simply conveyance of storm water from one location to another.

In summary, the additional investigations/designs necessary to include in subsequent work if a tunnel is included in the project are:

- Alignment determination
- Geologic literature review
- Bedrock seismic investigation
- Additional rock coring investigation
- Groundwater investigation
- Soil/Rock property investigations
- Tunnel lining design
- Inlet-outlet structure location and design.

3 Risk Issues for Tunnel Elements

3.1 General Risks

The same risks associated with above-ground structures apply to tunnels with respect to cost-growth, funding streams, labor and supply uncertainties; however the impacts of some non-natural risks are avoided in underground construction.

Traffic is not a significant factor in risks associated with tunnel construction, as the construction area is largely isolated. Weather is also not a factor for the same reason. Crime – theft and vandalism is also significantly less an issue with underground construction because the site is isolated and access is by necessity controlled.

3.2 Geologic and Geotechnical Risk

The geologic and geotechnical risks associated with tunnel components are similar to those of surface and near-surface features of the TSP, with regard to their actual risk – the major risk being inability of the system to function properly, resulting in localized flooding. The hazards causing the risks include:

- Unforeseen geologic features during construction (low to moderate probability)
- Regional seismic hazards (high probability)
- Local seismic hazards (high probability)
- Sudden hydraulic loading (high probability)

Unforeseen geologic conditions whether related to soil, bedrock or groundwater conditions may impact project cost, schedule and pose design challenges. These hazards are best mitigated by a more comprehensive geologic investigation along tunnel alignments, integrated with the existing data already available. The potential for unforeseen geologic and geotechnical conditions is considered moderate to low, based upon the extensive geotechnical investigations done to date and the availability of geologic data in the area. Much of the data specific to the alignments of flood control structures has been limited to soil conditions, however and additional data with respect to bedrock geology would be needed to further mitigate the uncertainty. Borings along the tunnel alignment on 500-feet centers to depths 5 tunnel diameters beyond the conceptual tunnel invert elevation (to allow for adjustment of the tunnel final design and to consider foundation conditions) would be appropriate.

Regional seismic hazards are well recognized in Southern California, and design criteria based upon the seismic risks of the area are a matter of regulation and professional standards. The impact of regional seismic activity would be limited to small-scale ground motion and shaking. Bedrock-based tunnel structures would be minimally impacted by regional seismic activity, and vertical shafts and inlet structures would be less susceptible than lateral features because of their vertical orientation, and ultimately they would be founded on the deeper bedrock, rather than potentially liquefiable or more mobile soil or fill. The potential for regional seismic hazards to is high but the potential to negatively impact tunnel related elements of the project is considered moderate to low, and would readily be mitigated by application of appropriate seismic design.

Local seismic hazards would include the presence of potentially active faults along the alignment of tunnel components. The location, geometric properties and historical record of activity of any such faults would have to be considered in the design of mitigation measures. At least 1 such feature has been identified along the potential alignment of a tunnel element, making this a high potential hazard. Likely mitigation measures would include both adjusting alignment to more favorably cross such features (where possible) and the design of a lining that could either respond flexibly with such movement or which would slip or deform in a manner that would limit areas requiring subsequent repair/maintenance. Resistance to mass rock movements along faults by tunnel lining is not practical.

Sudden hydraulic loading is a highly probable occurrence in a storm water tunnel, as the tunnels are typically not full and would only be loaded during high volume runoff events, and these events occur seasonally in Southern California. The design of the hydraulics of storm water tunnels is a relatively well understood discipline and is readily performed using both empirical methods and digital models in conjunction with surface hydrologic models that are and will be used for other aspects of the project.

RISK ISSUE	Probability	Consequences	Mitigation Difficulty
Unforeseen Geologic/Geotechnical feature	Moderate to Low	High	Low
Regional Seismic	High	Low	Low
Local Seismic	High	High	Low
Sudden Hydraulic Loading	High	Low	Low

Table G3-1: Summary of Hazard and Risk Mitigation Issues for Tunnel Elements

Appendix G – 3

Fragility Analysis without Project

1. Summary of Fragility Analysis

The fragility analysis without project is summarized in Table 1. After flood events there have been numerous work orders to repair the interior of the levees. Examples of these repairs include repair to rock slope protection (Drawing C02-101-SM-Fac, Sep 1994; Drawing C04-101-21M, 11Jul 1996; Drawing C05-101-6R Sep 1993) as well as slope repair. In addition to loads from flooding the levees are loaded by backwater from high tides. The coincidence of even a regular occurrence flooding event and high tides leads to regular loading to 50% of the levee height.

The foundation materials below the levees, which include consist of organic soils, silty sands, and sand are erodible. The coincidence high tides and high recurrence interval flooding increases the duration that the levees are subjected to these gradients.

Based on methodology discussed in this Appendix, zero probability of failure was assigned to the levee toe, 15% probability of failure was assigned to 50% of the levee and 85% probability of failure was assigned to 90% of the levee. Though there may be some differences between the left and right bank, they were treated as equivalent for the purpose of this analysis. Right and left banks are in the direction of storm drainage flow, which is generally towards the west.

Impact Area Code	Index Station from RAS	Bank on which the Impact Area Lies as moving downstream (Right, Left, or Both).	Is the reach/ impact area leveed, or not. If not, you only need to pull the "crest" elevation for the top of bank.	Bottom of channel elevation.	Top of levee or top of bank elevation.	Ground elevation on the interior side of the levee.	Toe elevation estimated from LIDAR data for use in the fragility curve	Zero probability of failure for the levee toe	The PNP (15% chance of failure) elevation for the fragility curve	15% probability of failure at the PNP	The PFP (85% chance of failure) elevation for the fragility curve	85% probability of failure at the PNP	Levee crest to be entered into HEC-FDA	Probability of failure at the levee crest in HEC-FDA (0.01 ft above the observed top of levee for modeling purposes)
Impact Area	**Station**	**Bank**	**Leveed (Y/N)**	**Invert**	**Crest**	**Terrain**	**Probable No-Failure Point Elevation (PNP)**	**Probability of Failure**	**Probable Failure Point Elevation (PFP)**	**Probability of failure**	**Probable Failure Point Elevation (PFP)**	**Probability of failure**	**Levee Crest (or top of bank)**	**Probability of Failure**
C05_6	75+93	Left	Y	0.7	12.9	5	5	0	6.19	0.15	11.72	0.85	12.9	1.00
		Right	Y	0.7	12.9	3	3	0	4.49	0.15	11.42	0.85	12.9	1.00
		Both	Y	0.7	12.9	3	3	0	4.49	0.15	11.42	0.85	12.9	1.00
C05_5	95+00	Left	Y	0.1	12.9	9.5	9.5	0	10.01	0.15	12.39	0.85	12.9	1.00
		Right	Y	0.1	12.9	7.5	7.5	0	8.31	0.15	12.09	0.85	12.9	1.00
		Both	Y	0.1	12.9	7.5	7.5	0	8.31	0.15	12.09	0.85	12.9	1.00
C02_1	52+88	Left	Y	1.9	11.9	10	10	0	10.29	0.15	11.62	0.85	11.9	1.00
		Right	Y	1.9	11.9	10	10	0	10.29	0.15	11.62	0.85	11.9	1.00
		Both	Y	1.9	11.9	10	10	0	10.29	0.15	11.62	0.85	11.9	1.00

Table 1. Fragility analysis without project.

2. Background

Figure 1 shows a map of the channels (CO2, CO4, C05, C06) and the impact areas (CO2_1, etc). The impact areas evaluated were C05_6, C05_5, and C02_1. The locations evaluated for impact area C05_6 and C05_5 were station 75+93 and 95+00, both located in reach 1. The location evaluated for impact area C02_1 was station 52+88.34, located in Reach 23, which is located in Channel 02. These analyses area based on design drawings and LIDAR data and consider pre-project (< year 2000) conditions. With project (>= year 2000) because there have been improvements to the channels since the year 2000 that are considered with project. Part of the challenge of this fragility analysis is to determine what the project looked like before improvements, which involves a review of past drawings to develop a chronology of channel construction in the areas of interest.

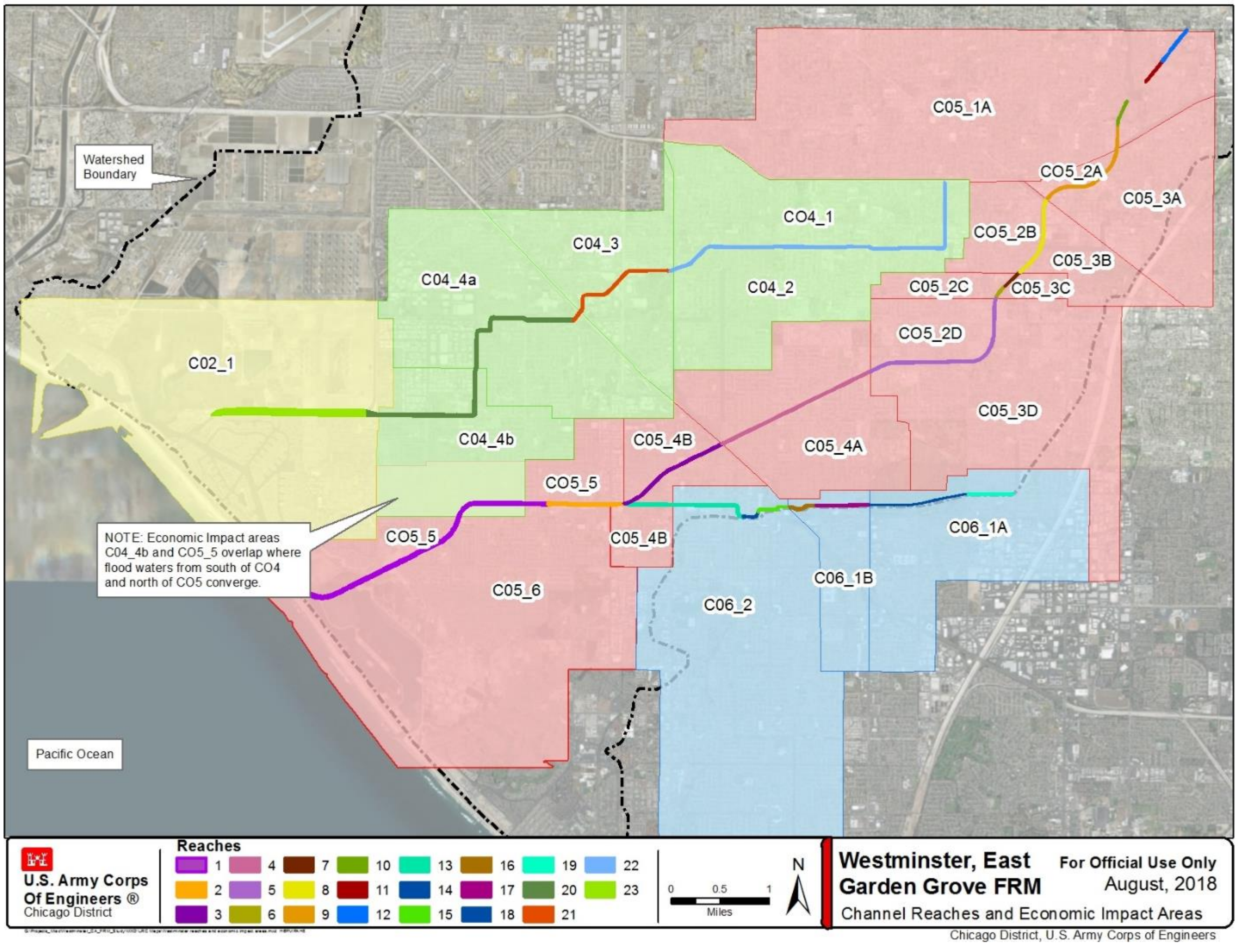


Figure 1. Map of channels C02, C04, C05, and C06 showing economic impact areas

As shown in Figure 2, Channel 05 was initially completed in September of 1959 and consisted mainly of earth trapezoidal channels lined with rip rap. In Sep 1993 a portion of reach 1 rip rap was repaired.

Figure 2. Pre-project impact area locations, reach 1

Figure 3 shows a typical section of channel C05, reach 1. Because the channels are sunk into the ground, the effective height of the levee is reduced. The section shown is a typical section, which is reflective of the sections analyzed for impact areas C05_6 (Sta 75+93) and C05_5 (Sta 95+00).

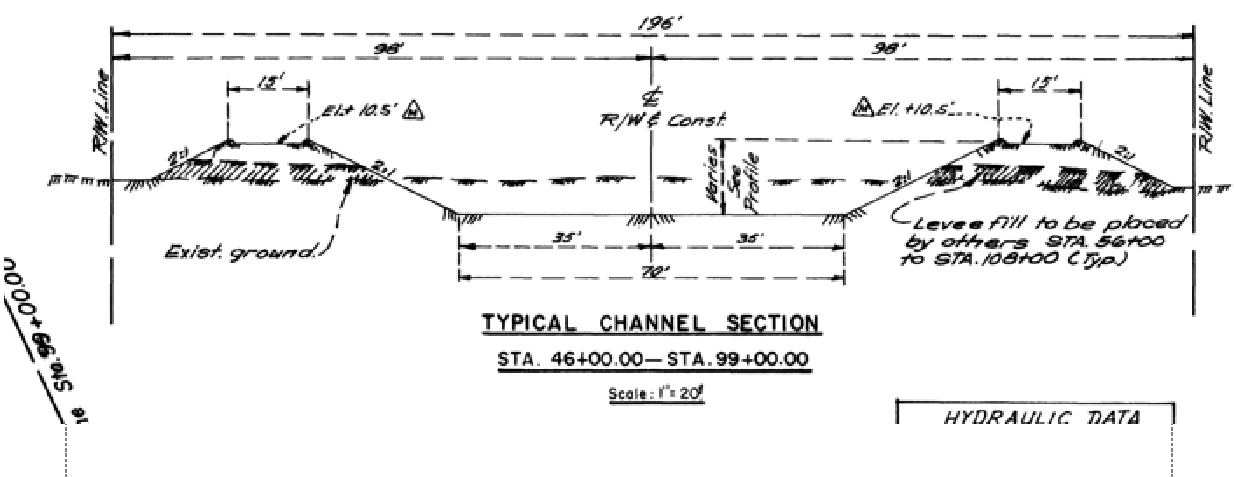


Figure 3. Typical section, Channel 05, Reach 1, without project

Figure 4 shows impact area C05_6 (Sta 75+93), the elevations were scaled off the design drawings and agree within 0.1 ft of the LIDAR data for the crest and 0.2 ft of the LIDAR data for the invert. However, these represent as built conditions, which date back to Sep 1959. The approximately 10 ft tall levees likely caused settlement of the underlying soils. The elevations were estimated based on hydraulic analyses. The LIDAR in this area reflects the with project condition.

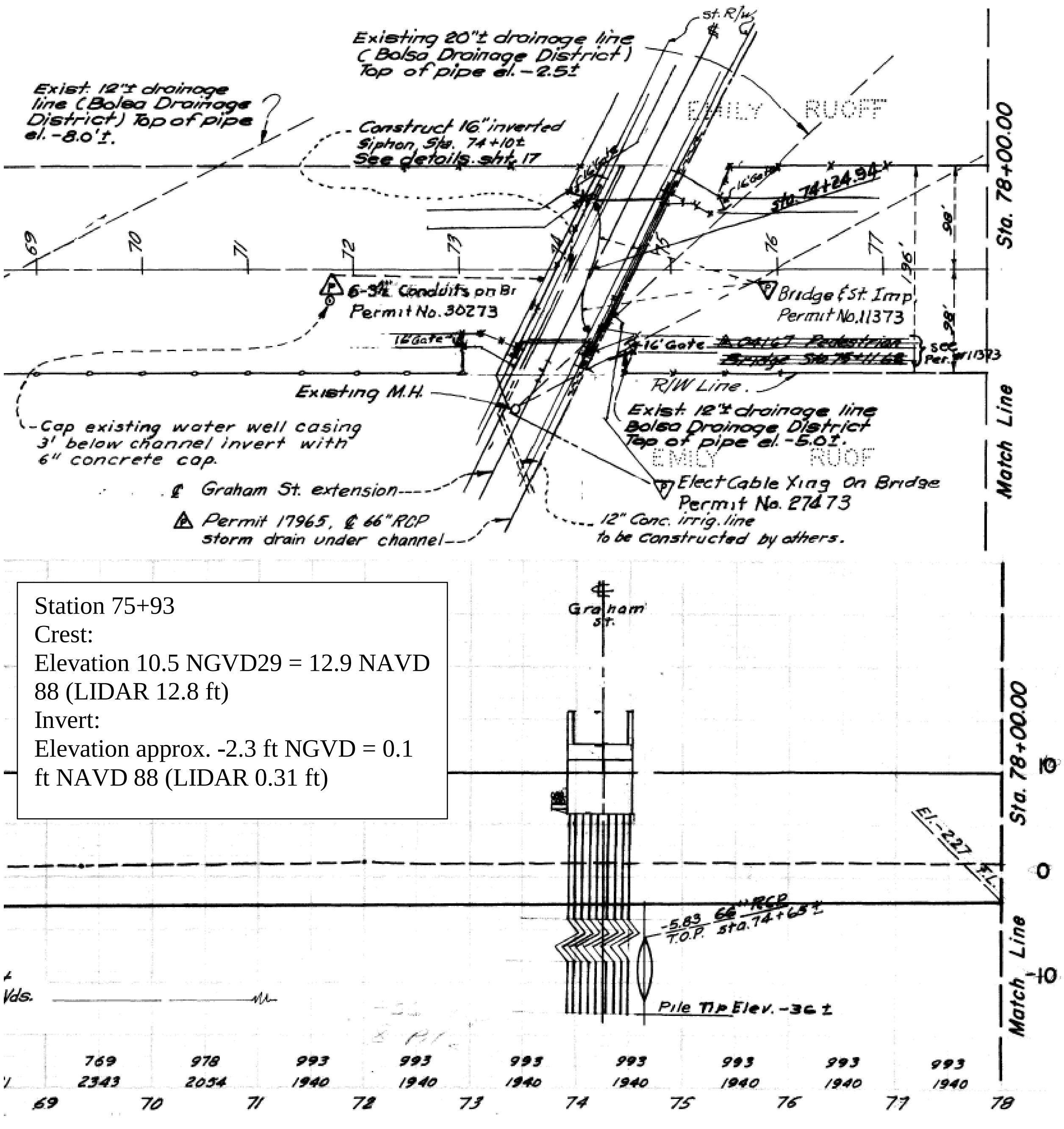


Figure 4. Plan view, Channel 05_6 (Station 75+93), Reach 1, without project

Figure 5 shows impact area C05_5 (Sta 95+00) the crest from the design drawings are within 0.1 ft of the LIDAR data and within 0.4 ft for the invert. However, as described for C05_6, the LIDAR data describes the with project condition in this area.

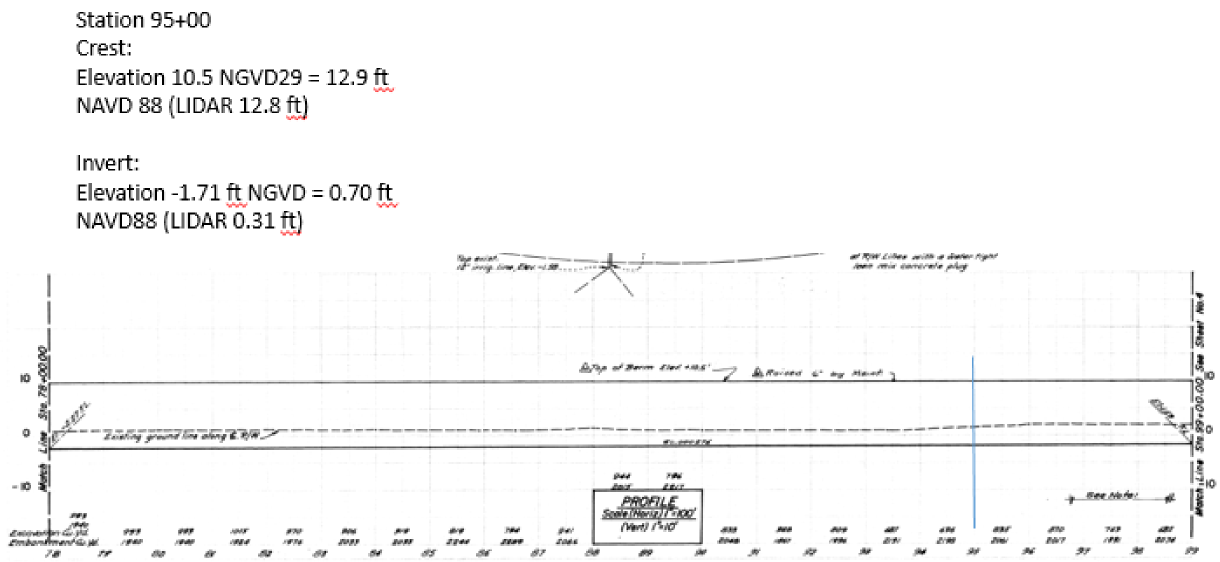


Figure 5. Plan view, Channel C05_5, Reach 1, without project

Though the levee is now with project south and west of Warner Avenue, an example of the levees for Channel 05, which would be how the levees south and west of Warner would have looked prior to project, are shown in Figure 6.

Figure 6. C05, Reach 1, Edwards looking west, example without project.

Figure 7 shows another view of the levees without project. In this case there is considerable erosion of the levee. Though analyses in this report will show that the gradients are low, there were no inspection reports available as part of this study. This adds to the uncertainty regarding the resilience of the levee system. There is, however, a history of repairs to the levees due to sloughing that coincides with flooding events.

Figure 7. C05, Reach 1, Edwards looking east, example without project.

Figure 8 shows the location of an investigation by Earth Mechanics (15 Oct 2009). This is from Station 101+00 to 151+25. Station 151+25 is very close to the end of reach 1. These investigations are used to evaluate the condition of the levees and foundations.

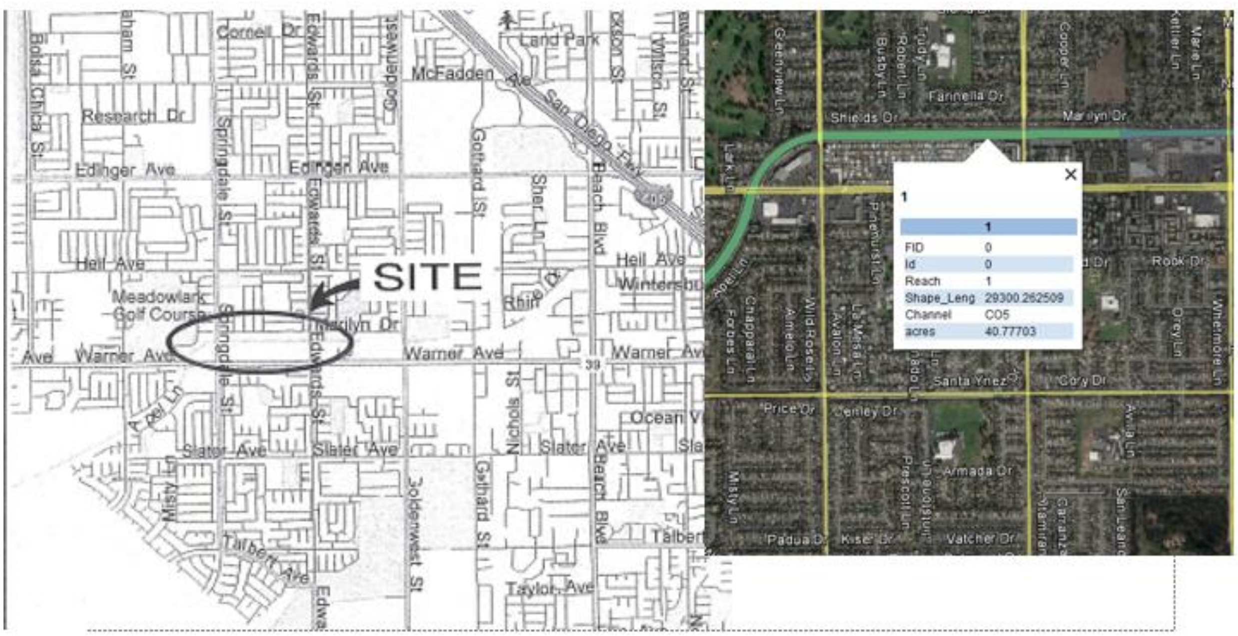


Figure 8. Location of geotechnical investigation by Earth Mechanics (15 Oct 2009)

A cross section developed by Earth Mechanics (15 Oct 2009), shown in Figure 9, illustrates clay levees to the left (north) and right (south) of the levees. Below the clay are organic soils and silts, which can be erodible.

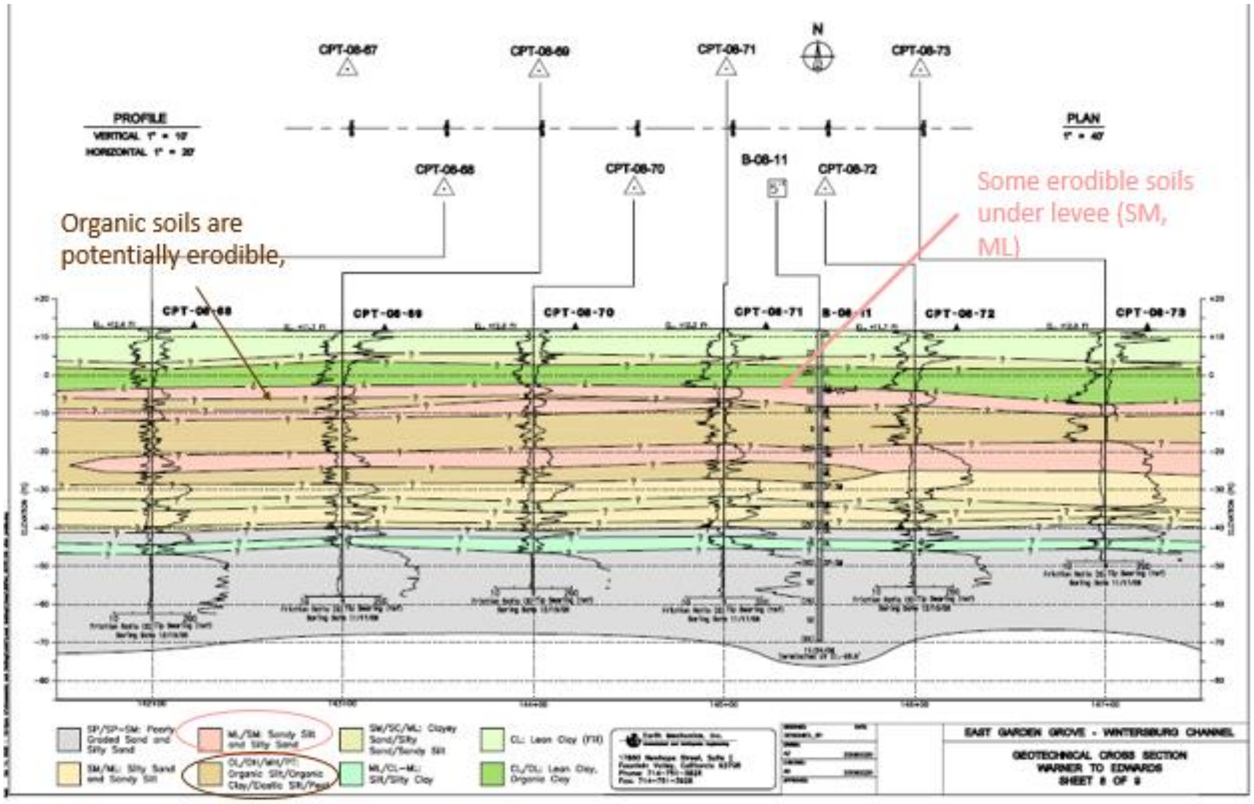


Figure 9. Geotechnical cross section developed by Earth Mechanics for Reach 1, to the east of Warner Avenue.

As shown in Figure 10, sheetpile was added to reach 1 south and west of Warner Avenue in 2008 (drawing C05-101-13R) and double sheetpile was added in this area in 2012 (Drawing C05-101-12). A review of Google Earth imagery shows that the C05-101-12 project drawings were implemented. However, because this work occurred after 2000, it is analyzed as “with project.”

Figure 10. With project impact area locations, reach 1

Figure 11 shows the project condition. From Warner Avenue and the C05 Channel (Reach 1), there is adequate protection against breach because of the double sheetpile walls on both sides of the channel. However, this is a portion of the channel system that is with project.


Figure 11. Warner looking southwest, example with project

Figure 12 shows upstream of Warner on the C05 channel there are stone lined trapezoidal channels, which represents the case without project. Plans to improve this channel are located in drawing C05-101-14, which are dated Apr 2010. However these plans have yet to be implemented.


Figure 12. Warner looking northeast, example without project.

Figure 13 shows calculations of elevations with project for impact area C05_06 (Sta 75+93). The elevation of 14.54 ft is above the C05 as built drawing and LIDAR data of 13.9 ft. The invert for the "as built" drawing (C05-101-13R) is 0.46 ft, which is 0.15 greater than the LIDAR data (0.31 ft).

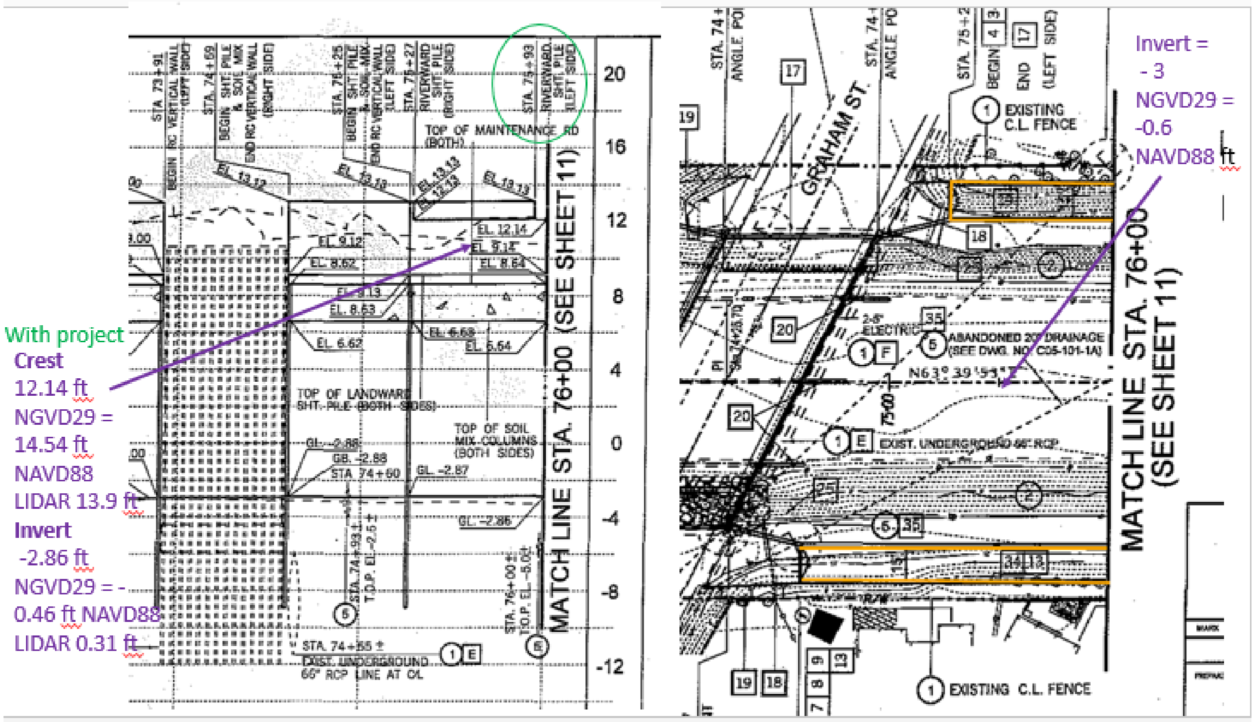


Figure 13. Impact area (C05_6), reach 1, crest and invert conditions with project

Figure 14 shows the with project condition. For C05, reach 1, there is existing sheetpile on the north side beginning at the pedestrian bridge.

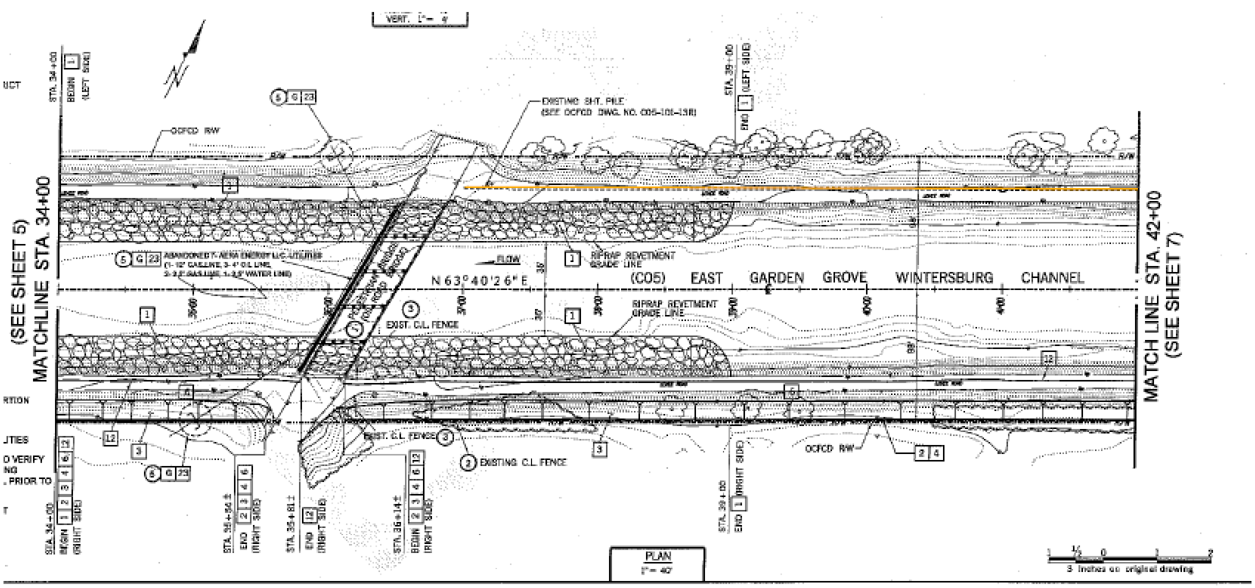


Figure 14. Reach 1, sheetpile begins at the pedestrian bridge on the north side. This portion of the project was completed 2 Feb 2008 (Drawing C05-101-13R), and is the with project condition.

Figure 15 shows the with project conditions for C05, reach 1. There is a sheetpile wall installed on the south side that begins at the pedestrian bridge.

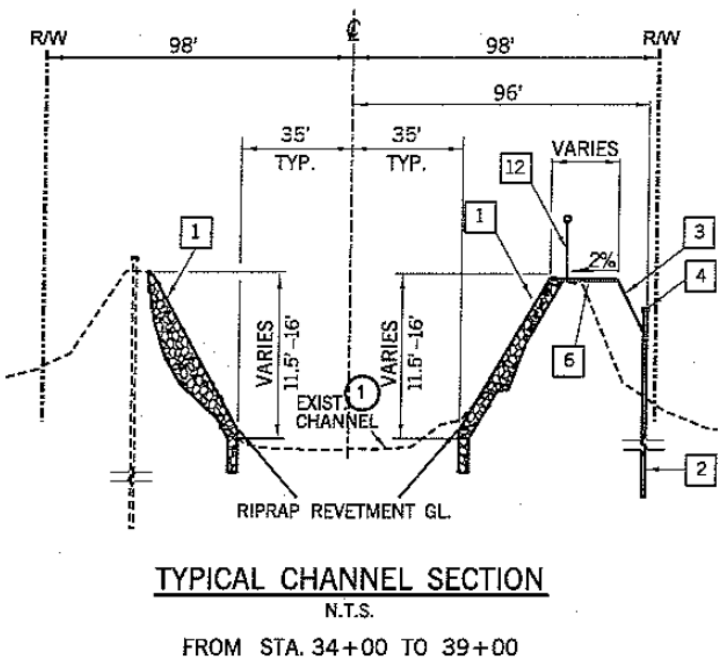


Figure 15. The south side has sheetpile installed that would reduce the likelihood of failure. This is with project

Figure 16 shows the with project condition for C05, reach 1, beginning at Station 53+16 there is a walkway and double sheetpile wall to the south side. This would provide significant protection against breach of the flood protection. There is a sheetpile wall on the north side that would also provide protection against breach.

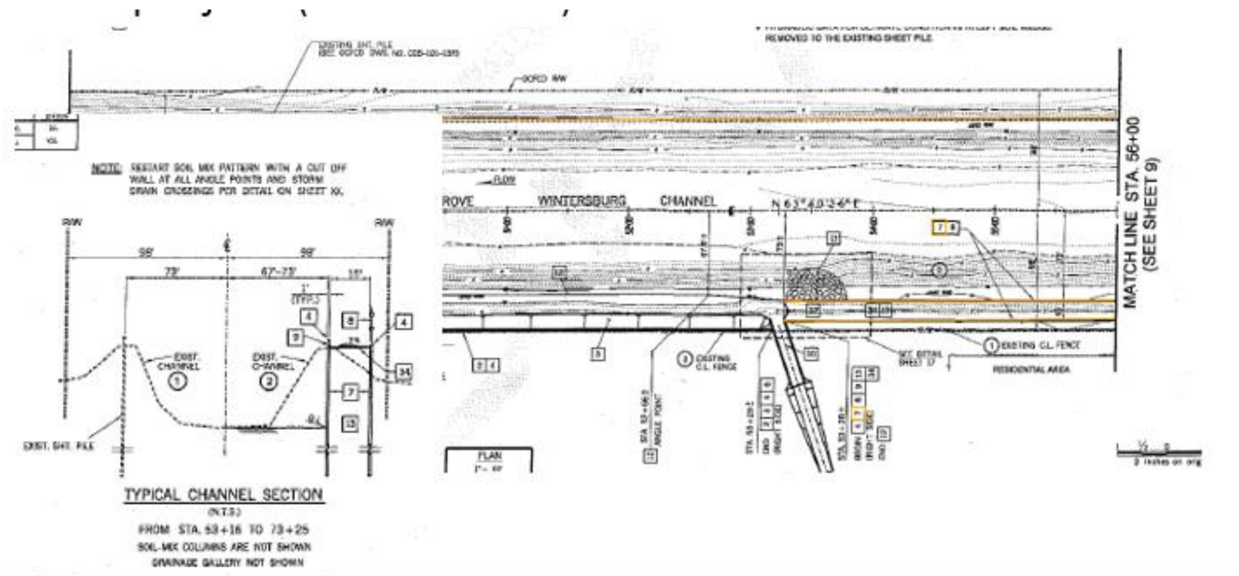


Figure 16. Double sheetpile begins at Station 53+16 on the south side, further reducing the likelihood of failure with project (Drawing C05-101-14).

Figure 17 shows C05, reach 1, with project, there is double sheetpile up until Warner Avenue, which is Station 102+02. At the station considered for the fragility analysis, which is 75+93, there appears to be very little chance of breach at the toe or the crest.

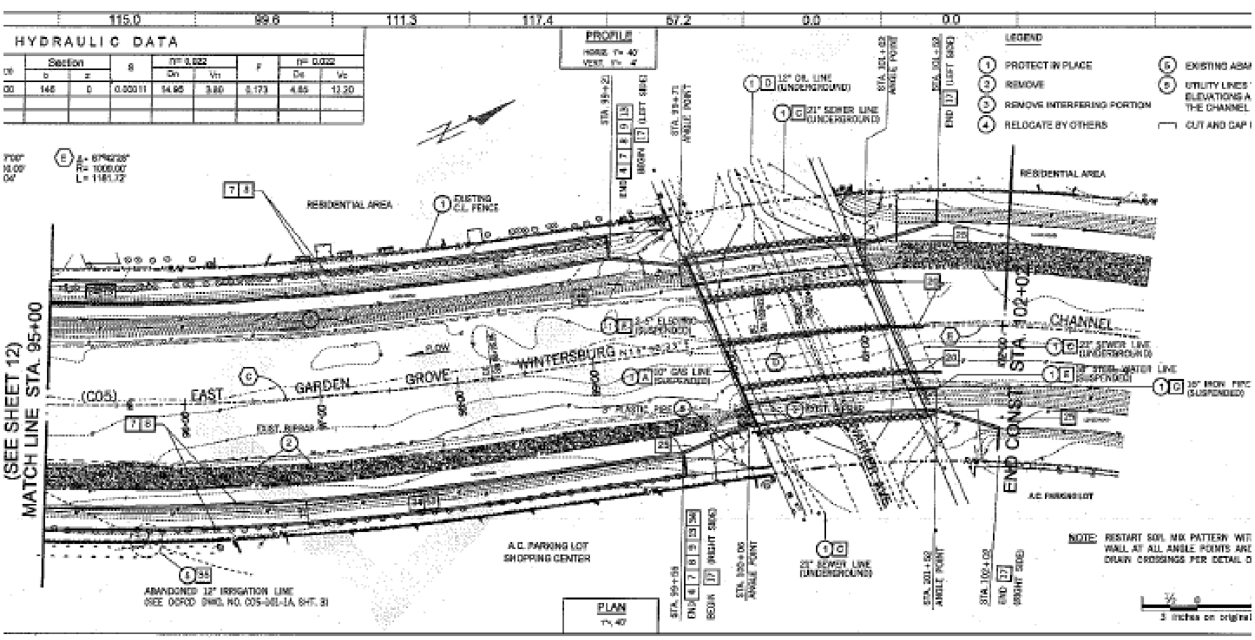


Figure 17. Channel C05, reach 1, with project in vicinity of the impact area point for C05_6

The drawings do not show improvements to C02, reach 23. Channel C02 consists of trapezoidal earth with rip rap lining as it did upon completion in Jun 1959. The construction history for channel C02, reach 23, is shown in Figure 18.

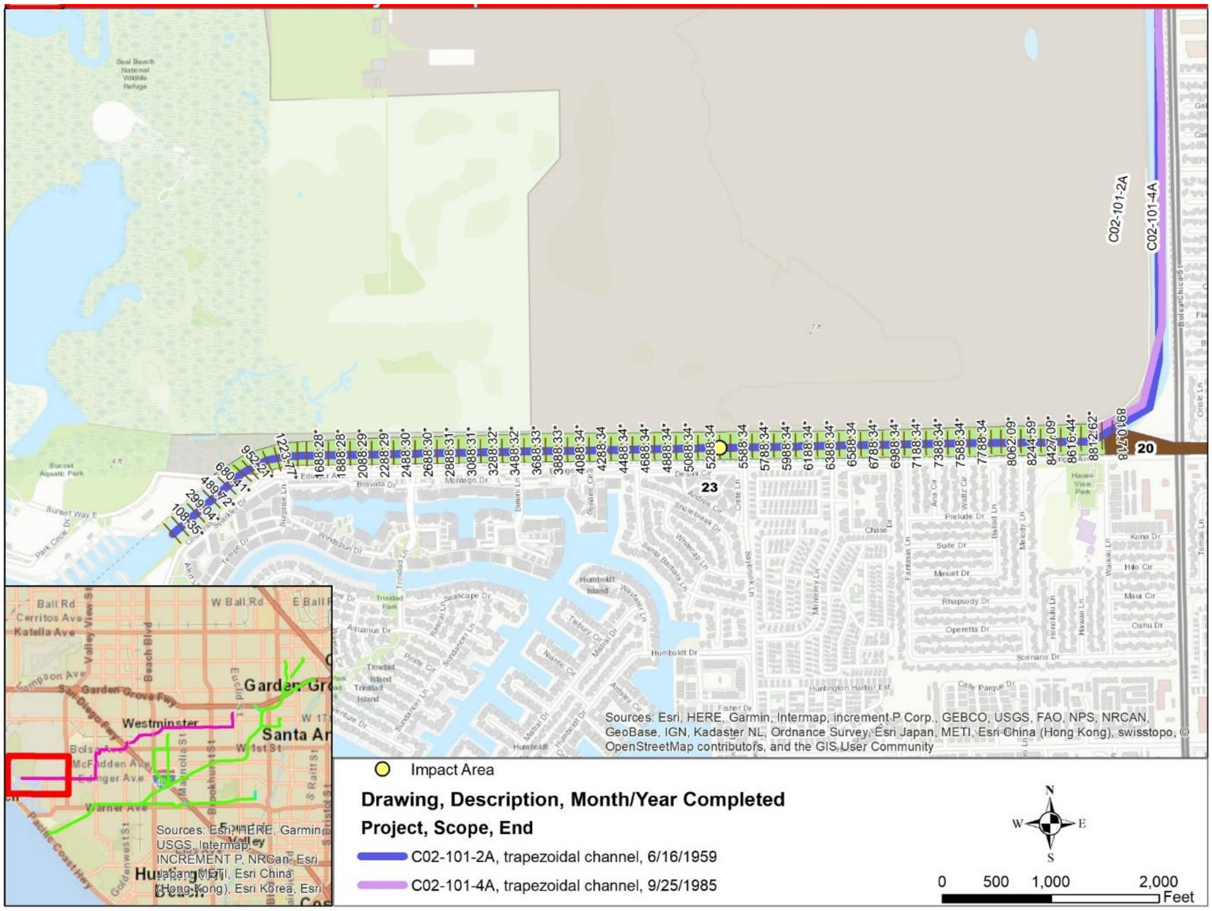


Figure 18. Pre-project impact area location, reach 23.

Figure 19 shows a typical section for C02 from the As Built drawings (C02-101-2A) which shows low levees and a trapezoidal ditch.

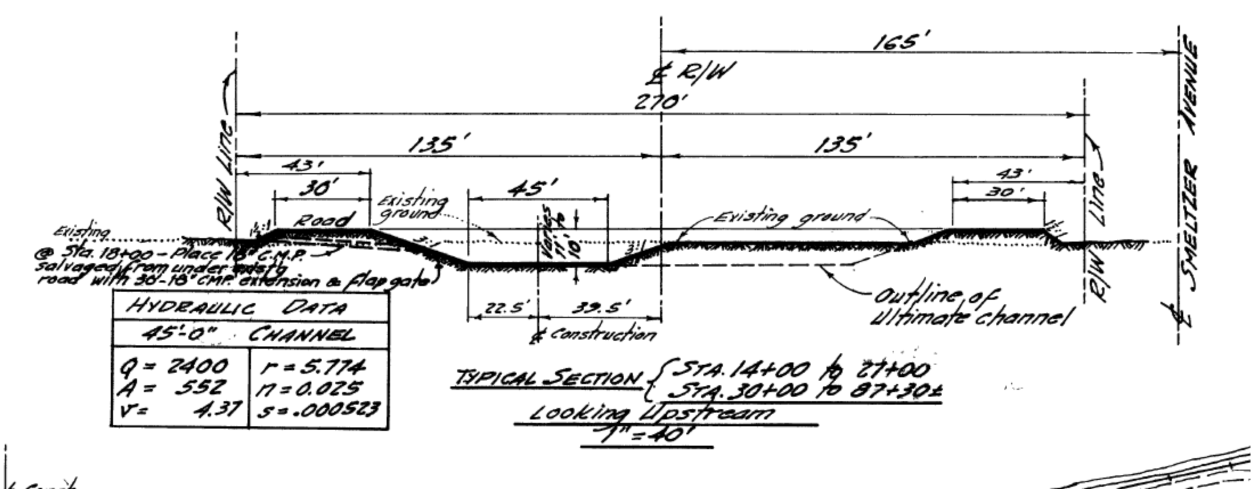


Figure 19. Channel C02, reach 23, without project, typical section.

Figure 20 shows the crest from the design drawings is 11.9 ft NAVD88, while the LIDAR shows 12.2 ft. However, there is a significant difference between the LIDAR invert (-9 ft NAVD88) and the design drawings 1.9 ft NAVD. Google Earth show the elevation to be 0 ft NAVD88. It is likely that the -9 ft is in error.

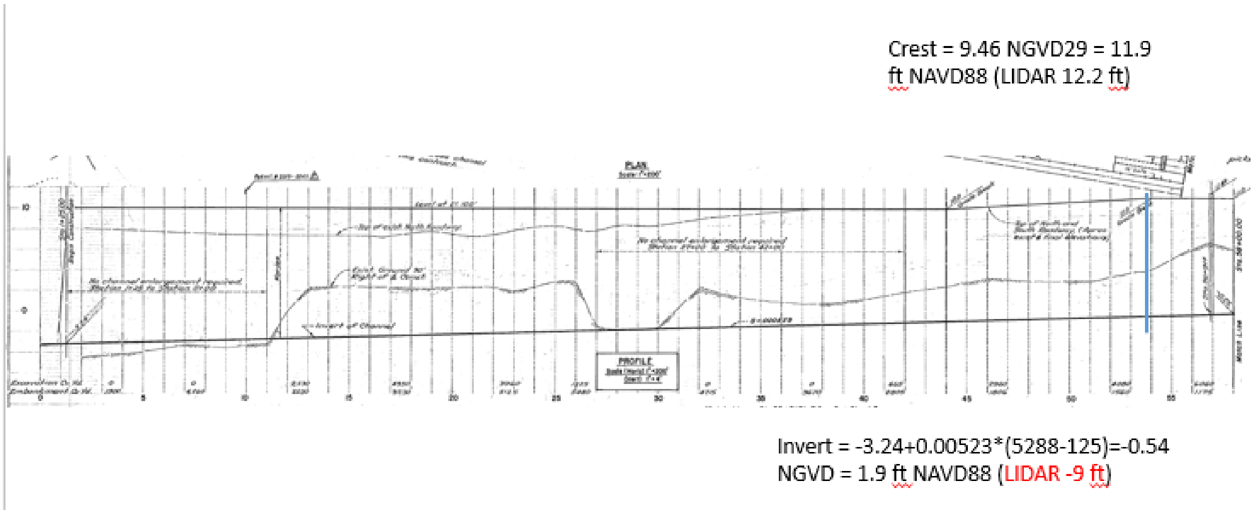


Figure 20. Channel C02, reach 23, Station 52+88, without project, plan view.

3. Gradient Analysis

C05_5

Conclusion

The analysis shows that failure may initiate at the point of the thinnest blanket with a channel water elevation of 7 ft. The loading of the levee is affected by tides, which might result in loading of 50% of the levee height, or approximately 6 ft, fifty percent of the time. Therefore the loading duration may be longer than the flashy nature of the channel flooding would initially indicate. However, for failure of the levee to occur, a boil would need to occur, mitigation such as a sand bag ring or filter drain would need to be unsuccessful, backward erosion piping would need to progress approximately 300 ft back to the levee, the levee would need to slough, and then overtop. Therefore, the likelihood of failure is reduced by at least one order of magnitude from 100% to 10%. Given the uncertainties of the analysis and rough nature of the planning guidance, which requires a 15% failure likelihood, the 15% failure probability is assigned to an elevation of 7 ft, which corresponds to roughly 50% of the levee loading.

The gradient at the toe of the levee is calculated to be 0.5 when the water in the channel reaches 11.6. The foundation soils are silty and therefore erodible. The piping distance is relatively short and the erodible soils are directly connected to the channel. Therefore the elevation that corresponds to a probability of failure of 85% is 11.6 ft, which corresponds to 90% levee loading.

Method

Figure 21a shows for the impact area C05_5 and a cross section drawn at Station 95+00 to evaluate gradients through the existing levee. Figure 21b shows a Google Earth image of this section. A seepage analysis was run for C05_5 using ground elevation available from LIDAR data and geology from the nearest centerline boring through C05, B-5, which was completed on 23 Jan 1959, and is located at Sta 95+30(+/-). Boring B-5 is shown in Figure 21c. The boring elevations were converted from NGVD29 to NAVD88 using the conversion: NAVD88=NAVD29+2.42. A cross section was then developed in GeoStudio (SeepW, 2012) as shown in Figure 21d. The channel boundary condition was varied from the top of the levee (el 12.75 ft) to 2 ft. The downstream side was assigned a flux boundary.   The levee and dark brown sandy clayey silt are assumed to act as a downstream seepage blanket, which ranges in thickness from 7.8 ft, 90 ft north of the centerline of the channel to 3 ft, 415 north of the channel centerline.

The gradient through the blanket is calculated at the toe of the levee and the thinnest blanket location (approximately 415 feet north of the channel centerline) as shown in Figure 21e.

The actual gradients, particularly at the thinnest portion, will likely be lower than calculated. This is because the model assumes that the blanket consists of a uniform layer or relatively impermeable material. In reality, there will be areas where pore water pressure can dissipate either through areas of higher permeability or penetrations such as light poles, telephone poles, swimming pools, etc. The criteria for sand boils, heavy seepage, etc. is provided in ETL 1110-2-569 (1 May 2005).

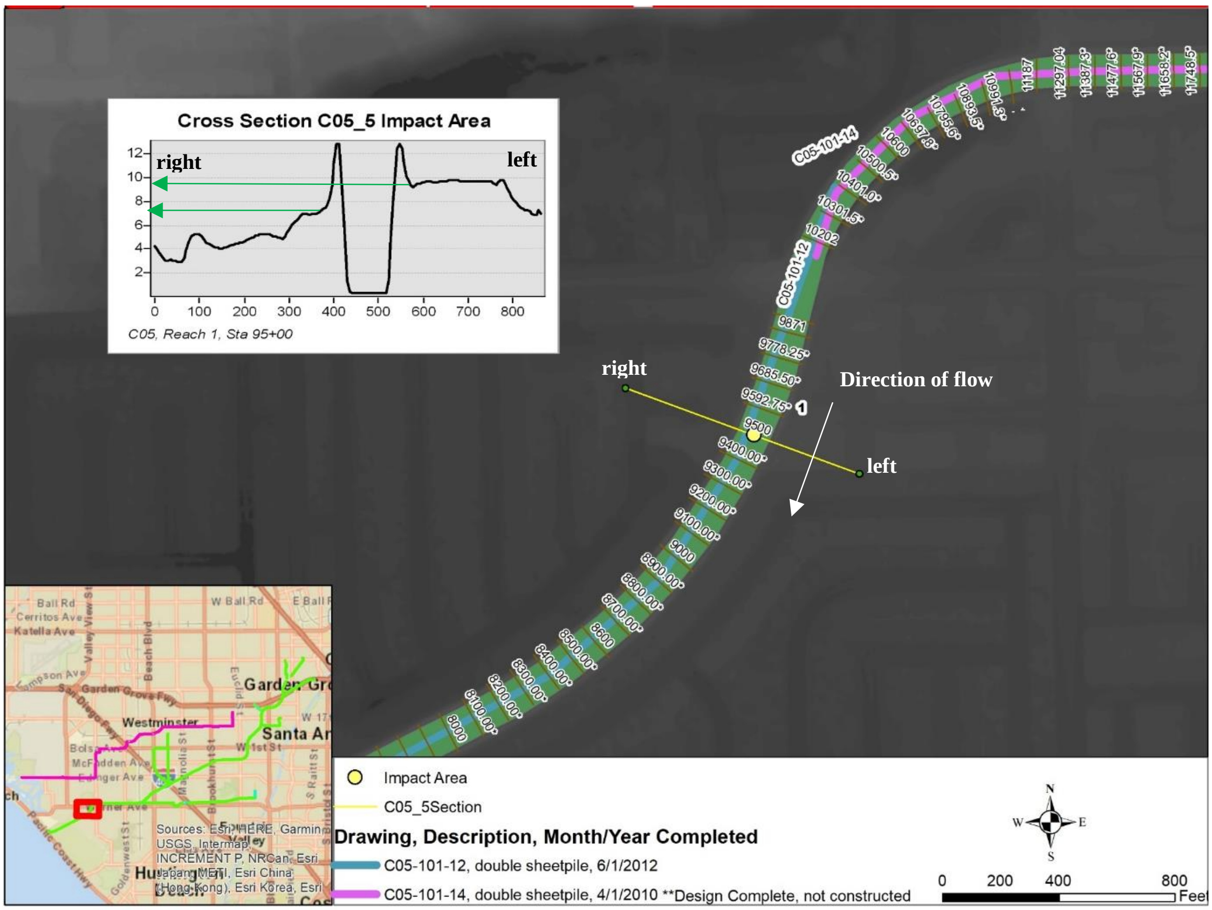


Figure 21a. Impact area C05_5, cross section location

Figure 21b. Google Earth image of area C05_5, cross section location

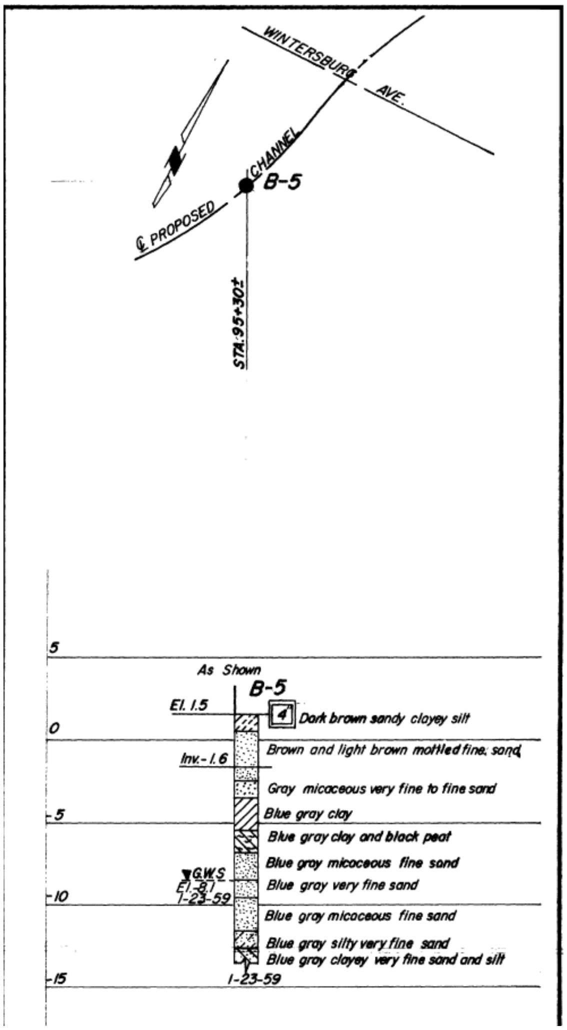


Figure 21c. Log for Boring B-5 used for section C05_5

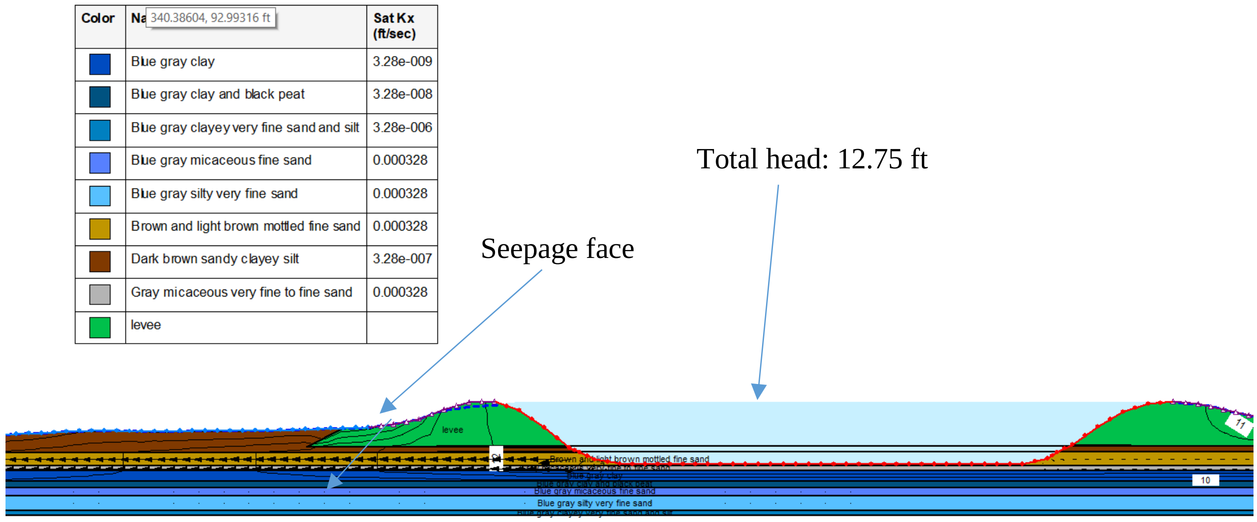


Figure 21d. SeepW (2012) cross section for seepage analysis at C05_5

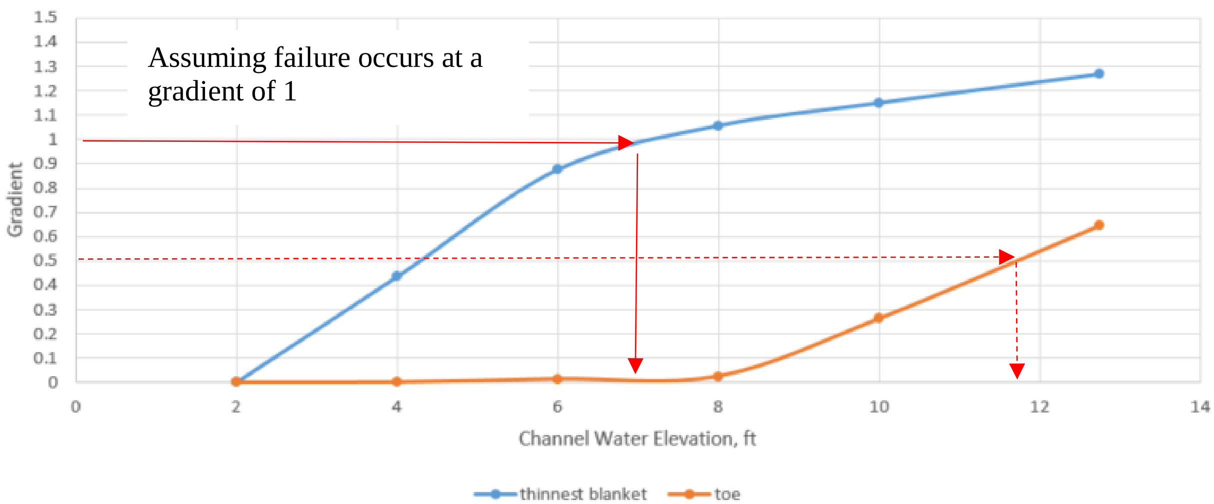


Figure 22e. Showing that at 7 ft channel head that the levee has failure conditions at the thinnest blanket, and that failure could initiate at the toe at a channel elevation of 11.6 ft.

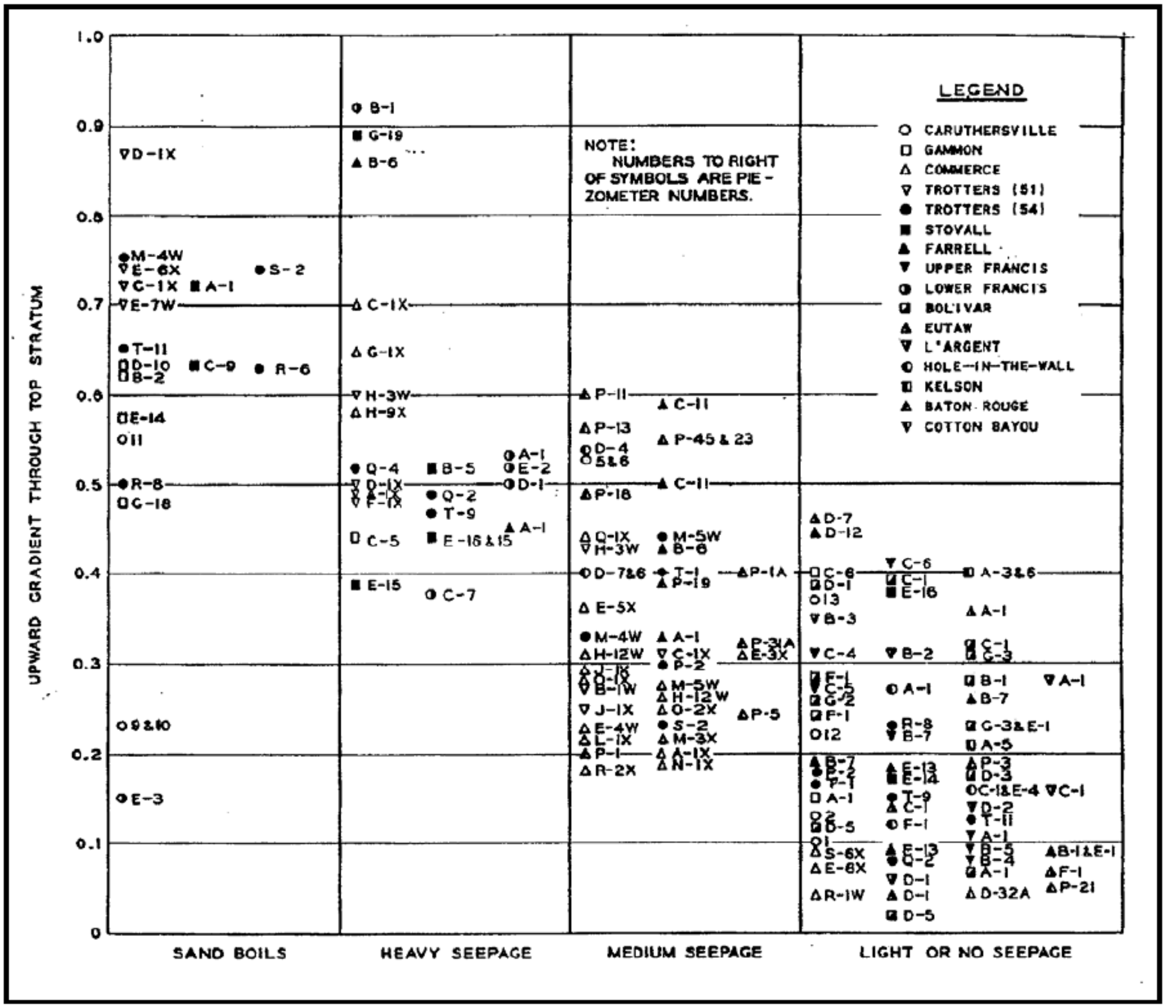


Figure 1. Severity of seepage as related to upward gradient through top stratum (from U.S. Army Engineer Waterways Experiment Station 1956, Vol. 1, Figure 47, p. 272)

Figure 22d. Sand boils, which are related to failure initiation have been seen in the range of gradients of 0.5 to 0.9.

C05_5 and C02_1

For C05_6, a cross section as shown in Figure 23, and For C02_1 as shown in Figure 24. For C05_6 and C02_1 the likelihood of failure is estimated based on the more detailed analysis of C05_5, which is a 15% probability of failure at 50% levee loading and an 85% probability of failure when the levee is loaded to 90%.

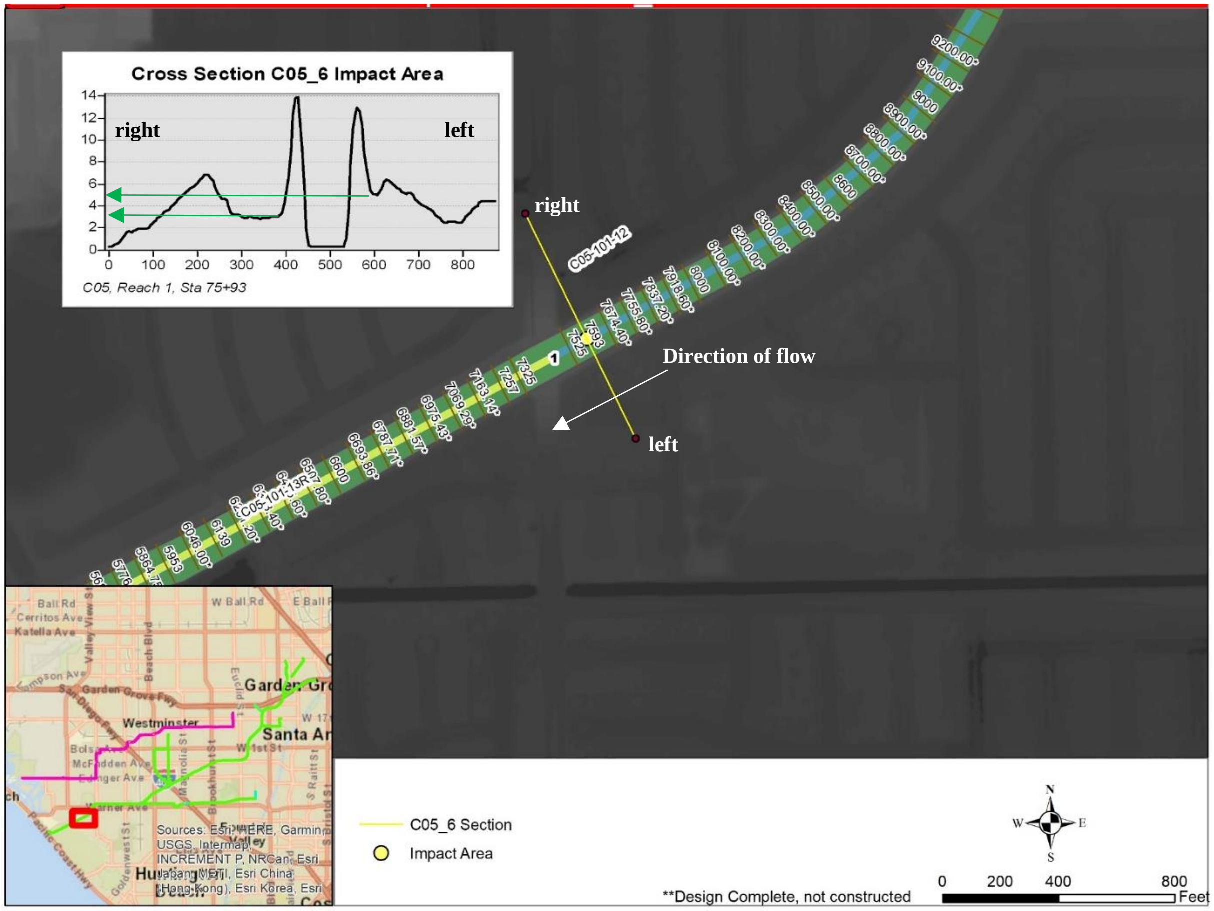


Figure 23. Impact area C05_6, Section Location

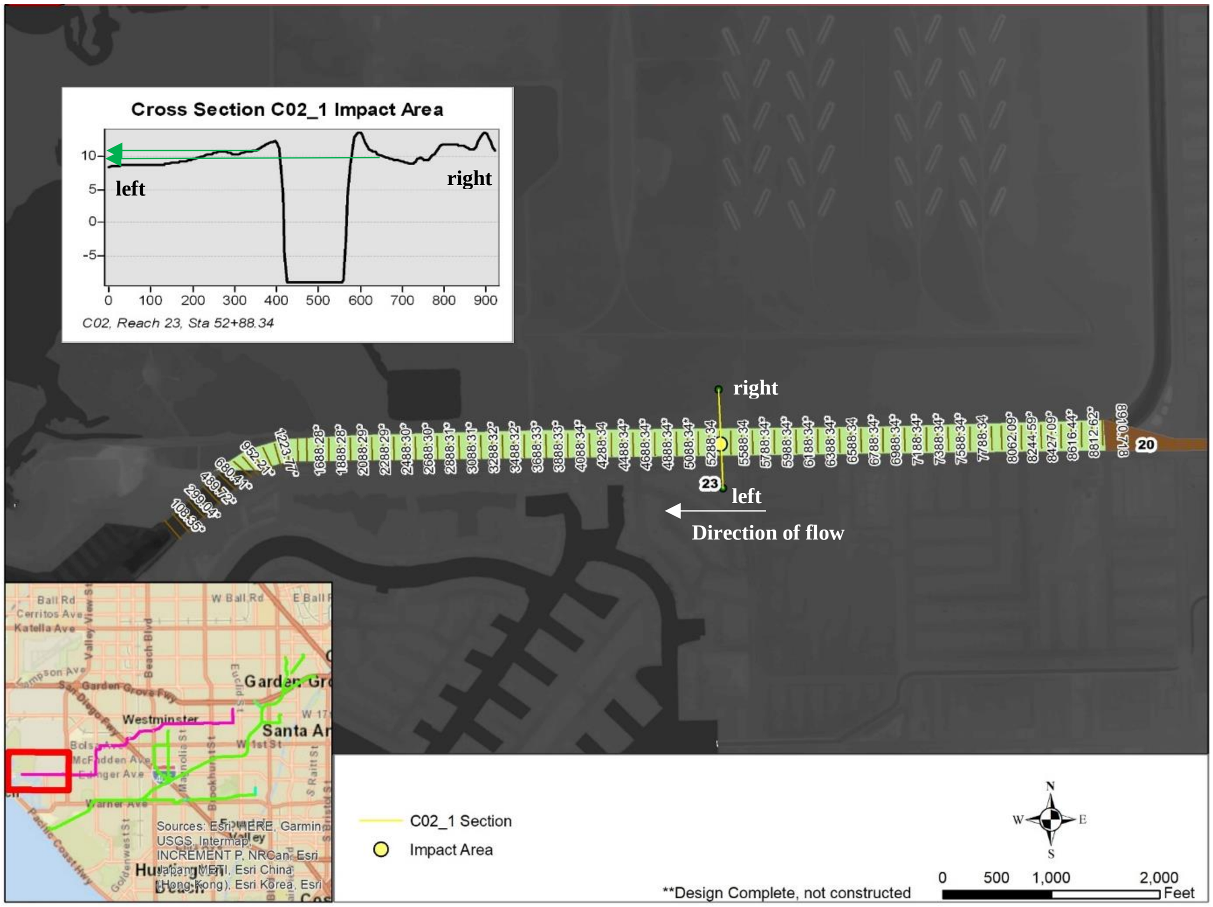


Figure 24a. Impact area C02_1, section location (bottom elevation of ditch based on LIDAR is in error)

Figure 24b. Impact area C02_1, section – Google Earth (checking bottom ditch elevation)

4. Slope Stability Analyses

Rapid drawdown was evaluated for Section C05_5. The section analyzed is shown in Figure 26. Generalized soil properties were chosen for rapid drawdown. Rapid drawdown was evaluated for various starting water levels to a lower water level of +2 ft NAVD 88 as shown in Table 2.

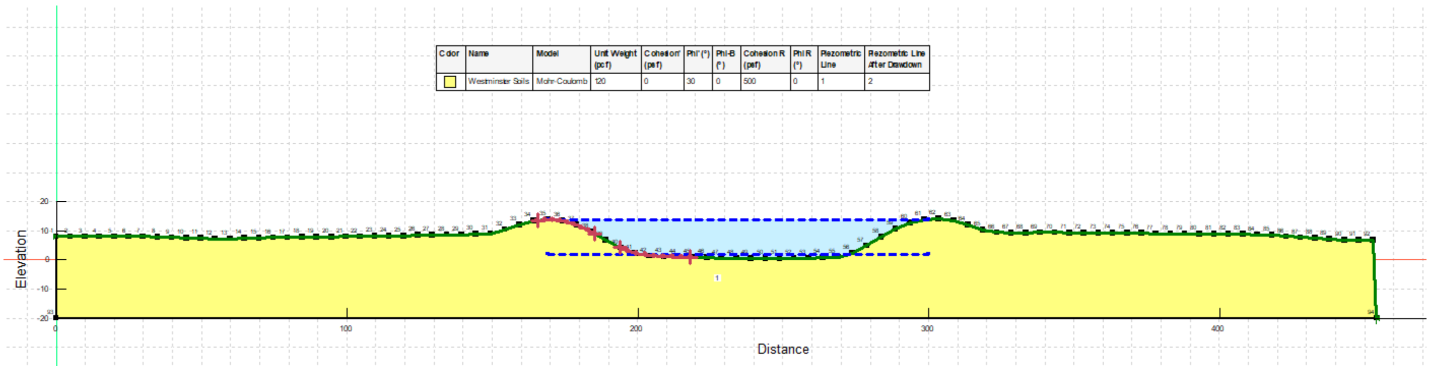


Figure 26. Typical rapid drawdown analysis section

Rapid Drawdown		P(f)	
WL (ft, NAVD88)	FS	III	IV
0	1	31%	71%
4	1.011	30%	70%
6	0.987	33%	71%
8	0.9	45%	68%
10	0.862	48%	63%
12	0.912	43%	69%
12.75		100%	100%

Table 2. Summary of starting water levels for rapid drawdown analyses, computed factors of safety and corresponding probabilities of failure assuming category III and category IV (poor) site investigation/design.

Category III: $P(f) = 2.5*FS^3 - 11.188*FS^2 + 12.963*FS - 4.7769$

Category IV $P(f) = 2.381FS^3 - 11.249FS^2 + 15.082FS - 6.3648$

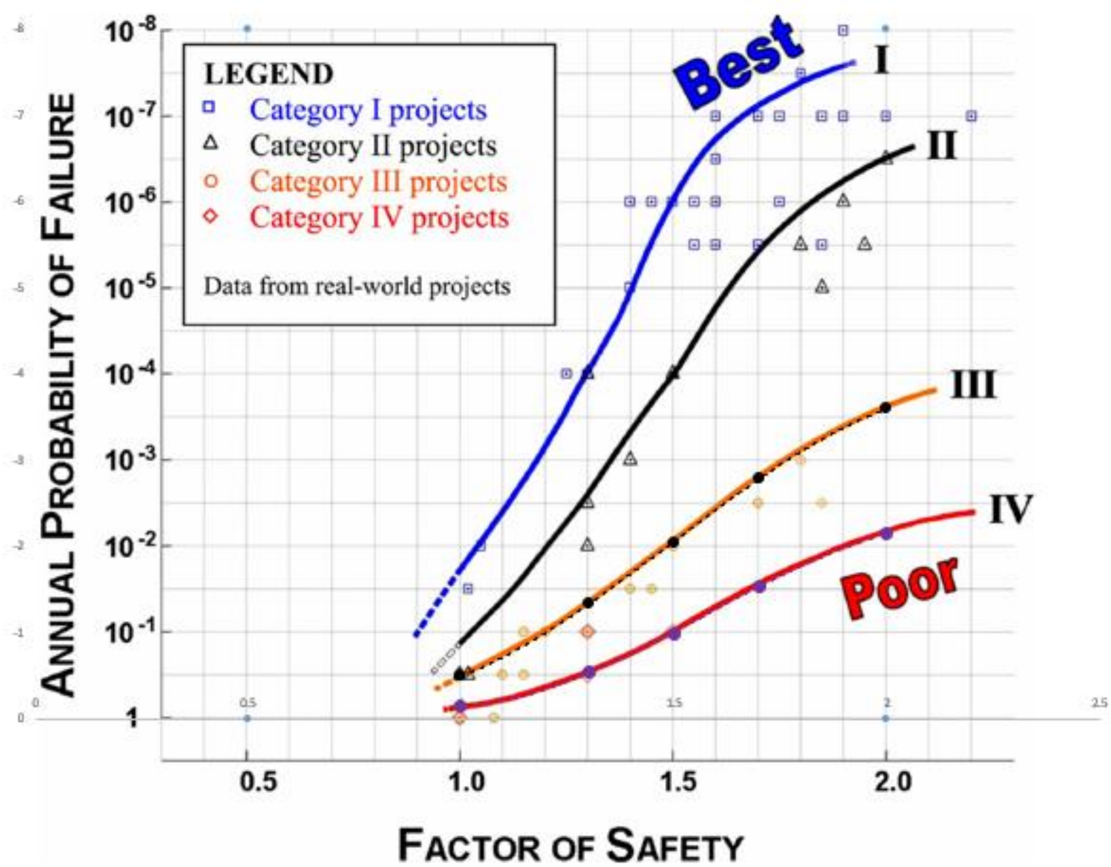


Fig. 1. Factor of safety versus annual probability of failure

Figure 28. Relating factor of safety to annual probability of failure (Lamb and Da Silva, …)

Da Silva and Lamb evaluated projects to relate the annual probability of failure to the corresponding factor of safety as shown in Figure 28. Functions were developed based on category III and category IV projects. Category IV projects are sites with little to no investigation and engineering design. The factor of safety is then compared to the annual probability of failure for these two categories and is plotted in Figure 29.

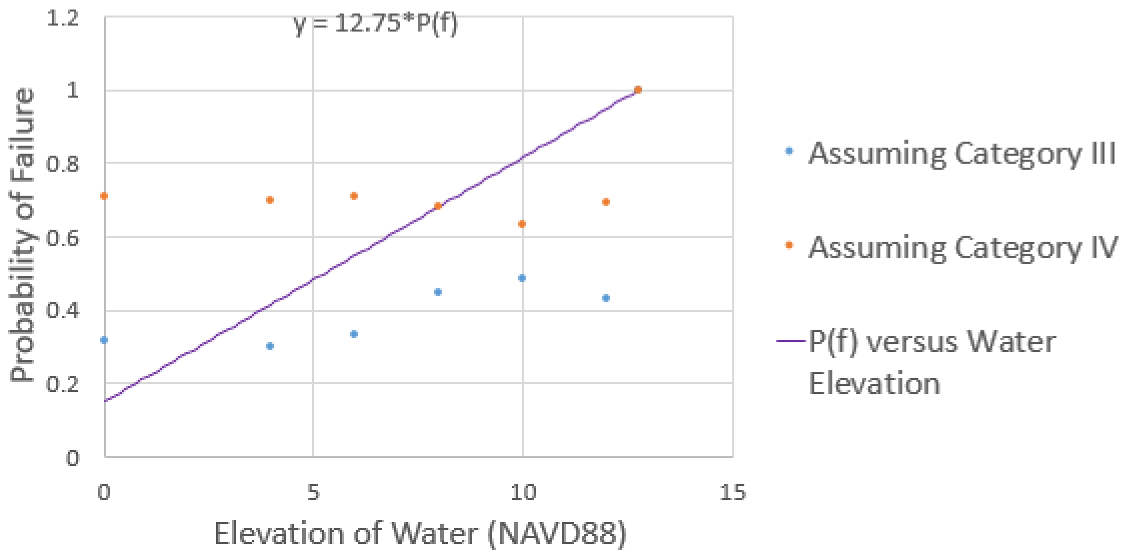


Figure 29. Relating water elevation to probability of failure assuming either Category III or Category IV projects

Figure 29 shows general trends. However a clear relationship is not shown. It is assumed at the crest the probability of failure is certain and a straight line relationship is estimated between water elevation and failure likelihood. This general relationship is then applied to each section to estimate the 15% and 85% probability of failure.

Because the ground elevation is often above the bottom of the drainage ditch, this linear interpolation between water elevation and probability of failure is applied from the landside elevation. For example, for C05_5 the protected side is at an elevation of 5 ft NAVD88 on the left side. The crest is 12.9 ft NAVD. Therefore the elevation corresponding to 15% probability of failure is:

$$Elev(P = 15\%) = [Elev(crest) - Elev(terrain)] * 15\% + Elev(terrain)$$

$$Elev(P = 15\%) = elevation\ where\ probability\ of\ failure\ is\ 15\%,$$

$$Elev(crest) = elevation\ of\ levee\ crest,$$

$$Elev(terrain) = elevation\ of\ protected\ side\ ground. The\ levee\ is\ above\ this\ elevation$$

5. Geotechnical Investigations

As shown in Figure 25, Diaz•Yourman – GeoPentech – Kinnetic Laboratories / Joint Venture surveyed the available drawings and reports in their feasibility appendix which was finalized July 27, 2016.

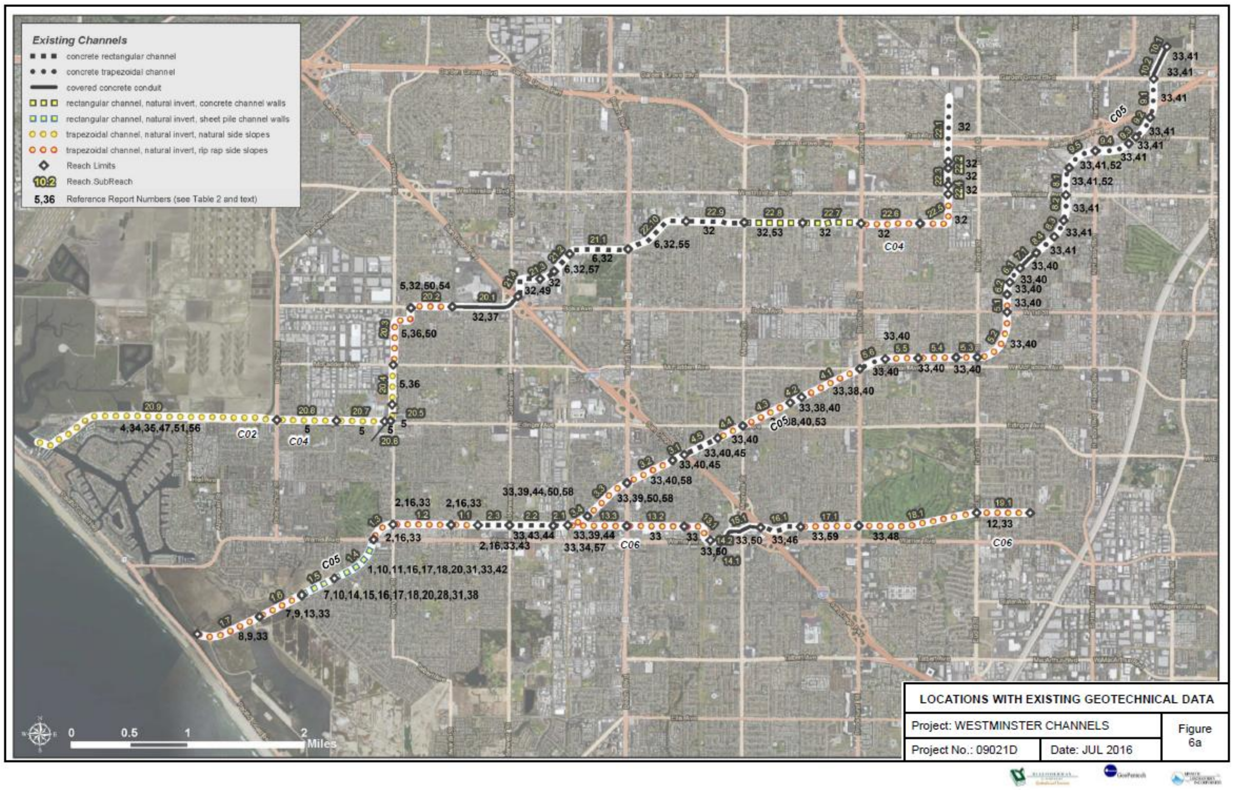


Figure 25. Summary of geotechnical report for the channel systems

Reports and plans applicable to reach 1 are summarized in Table 2. Reports and plans applicable to reach 23 are summarized in Table 3.

Diaz Yourman GeoPentech Kinnetic Laboratories Number	Reach	Author	Type	Borings	Channel	Description	Sta	Date	Report
1	1	Earth Mechanics	Geotechnical investigation report		C05	Graham to Warner	75+00 to 100+00	15-Oct-09	Earth Mechanics, Inc. 15 Oct 2009. Final geotechnical investigation report, East Garden Grove – Wintersburg Channel improvements from Graham Street to Warner Avenue (Sta 75+00 to 100+00) Huntington Beach, Orange County, California. Prepared for Orange County Public Works, Materials Laboratory, 1152 E. Fruit Street, Santa Ana, CA 92702. EMI Project No. 08-155.
2	1	Earth Mechanics, Inc.	Geotechnical investigation report		C05	Warner to upstream of Edwards Street	101+00 to 151+25	15-Oct-09	Final Geotechnical Investigation Report, East Garden Grove-Wintersburg Channel Improvements from Warner Avenue to Upstream of Edwards Street (Station 101+00 to 151+25), Huntington Beach, Orange County, California, 717 pp.
7	1	Orange County Resources and Development Management Department	Plans		C05	3800 ft downstream of Graham to Graham		1-Oct-07	City of Huntington Beach. Oct 2007. Plans for construction of East Garden Grove – Wintersburg Channel, north levee emergency project. Facility No C05 from 3800 feet downstream of Graham St to Graham St. Funded and Maintained by Orange County Flood Control District.
8	1	Spencer et al.	Slope stability analysis		C05	North levee downstream of oil bridge		23-Sep-08	Spencer, J. 23 Sep 2008. East Garden Grove – Wintersburg Channel OCFCD Facility C05, slope stability analysis of the north levee downstream of the oil bridge, fall 2008.
9	1	URS	Geotechnical investigation report		C05	Proposed sheet pile buttress support	Sta 34+00 to Sta 53+16	18-Jan-11	URS 18Jan2011. Geotechnical investigation report for East Garden Grove – Wintersburg Channel (OCFCD Facility C05) improvements phase 1, proposed sheet pile buttress support from Sta 34+00 to Sta 53+16, City of Huntington Beach, Orange County, California. Prepared for Orange Public Works, Materials Laboratory 1152 E Fruit Street, Santa Ana, CA 92701. Project 29871517
10	1	Hushmand Associates	Geotechnical investigation report		C05		37+00 to 102+00	20-May-10	Hushmand Associates, Inc. 20 May 2010. Geotechnical Investigation East Garden Grove-Wintersburg Channel (C05) Levee Soil Mix Project Groundwater Impact Evaluation Station 37+00 to Station 102+00 Huntington Beach, Orange County, California HAI Project No. COO-09-001. Prepared for County of Orange, Materials Laboratory Resources and Development Management Department 1152 E. Fruit Street, Santa Ana, CA 92701
11	1	Earth Mechanics, Inc.	Geotechnical investigation report		C05			15-Oct-09	Final Geotechnical Investigation Report, East Garden Grove - Wintersburg Channel Improvements from Graham Street to Warner Avenue (Station 75+00 to 100+00), Huntington Beach, Orange County, California, 343 pp.
12	1	Earth Mechanics	Geotechnical report		C05			5-May-08	Earth Mechanics, Inc. 5 May 2008. Deep soil mix column levee structure, East Garden Grove-Wintersburg Channel Improvement, Huntington Beach, Orange County, California. EMI Project No. 07-133
14	1	Advanced Earth Systems	Geotechnical evaluation		C05	review and feasibility evalution proposed levee improvements	48+00 to 74+25	29-Jun-05	Geotechnical review and feasibility evalution proposed levee improvements East Garden Grove - Wintersburg Channel Station 48+00 to 74+25 (C05). Prepared for County of Orange Resources & Development Management Department 1152 East Fruit Street, Santa Ana CA 92701. AES Project No.: 02-122-3
2	1	Earth Mechanics, Inc.	Geotechnical investigation report		C05	Warner to upstream of Edwards Street	101+00 to 151+25	15-Oct-09	Final Geotechnical Investigation Report, East Garden Grove-Wintersburg Channel Improvements from Warner Avenue to Upstream of Edwards Street (Station 101+00 to 151+25), Huntington Beach, Orange County, California, 717 pp.
16	1	Smith et al.	Geotechnical investigation report		C05		50+00 to 152+00	Mar-01	Smith, K.E., Sterling, J.D, Martindale, S.G., Leiby, D.A, and J. Eaton. Mar 2001. Geotechnical investigation materials report East Garden Grove Wintersburg Channel C05 Station 50+00 to Station 152+00. Project Number EF03515.
33	1	Williamson & Schmid	Project report					1-Dec-1994	Project Report for East Garden Grove - Wintersburg (C05) and Oceanview (C06) Channels, 279 pp.
15	1	MACTEC Engineering and Consulting, Inc.	Geotechnical investigation report					12-Feb-04	Final Report of Geotechnical Investigation, Proposed East Garden Grove Wintersburg Channel C05 Improvement, Southwest of Graham Street, Southern Levee, from Station No. 48+00 to Station No. 74+25, Huntington Beach, California, 275 pp.
16	1	Smith et al.	Geotechnical investigation report		C05		50+00 to 152+00	Mar-01	Smith, K.E., Sterling, J.D, Martindale, S.G., Leiby, D.A, and J. Eaton. Mar 2001. Geotechnical investigation materials report East Garden Grove Wintersburg Channel C05 Station 50+00 to Station 152+00. Project Number EF03515.
17	1	WRC	Review		C05	Review of water quality enhancement and perched water buffer components		23-May-12	WRC Consulting Services, Inc. 23 May 2012. Review of water quality enhancement and perched water buffer components for East Garden Grove – Wintersburg Channel (C05) levee improvements. Reference; Agreement D07-067 Work order No. EF03531.
18	1	Spencer et al.	Geotechnical evaluation		C05	North Levee downstream of Graham Street		25-Sep-07	Spencer, J., Linger, P., Fayad, A., Jones, P., and N. Majaj. 25 Sep 2007. East Garden Grove – Wintersburg Channel OCFCD Facilitiy #C05. Quantitative engineering analysis of North Levee downstream of Graham Street – Fall 2007. County of Orange, Resources Development Management Department on behalf of Orange County Flood Control District.
20	1	MACTEC Engineering and Consilting, Inc.	Geotechnical investigation report					12-Feb-04	Report of Geotechnical Investigation, Proposed East Garden Grove Wintersburg Channel C05 Improvement Southwest of Graham Street, Northern Levee From Station No. 48+00 to Station No. 74+25, Huntington Beach, California, Main Report and Appendix A, 55 pp.
28	1	Pacific Soils Engineering, Inc.	Geotechnical investigation preliminary					2-Feb-98	Preliminary Geotechnical Investigation, Proposed Residential Development, Tentative Tract 15377, City of Huntington Beach, California, and Tentative Tract 15419, County of Orange, California, 264 pp.
31	1	County of Orange	Plans		C05	Bolsa Chica Tide Gates to Upstream Warner Ave	6+34 to 102+02	Jul-10	County of Orange OC Public Works Department. Jul 2010. Plans for construction of East Garden Grove – Wintersburg Channel, O.C.F.C.D. Facility No. C05 from Bolsa Chica Tide Gates to upstream of Warner Avenue (From Station 6+34 to Station 102+02). 90% Plan Submittal.
33	1	Williamson & Schmid	Project report					1-Dec-1994	Project Report for East Garden Grove - Wintersburg (C05) and Oceanview (C06) Channels, 279 pp.
38	1	Orange County Environmental Management Agency	Plans					3-Jan-1992	Plans for Construction of Westminster Channel, Facility No. C04, from U/S of Magnolia to D/S of Brookhurst, 12 pp.
42	1	Orange County Environmental Management Agency	Plans					9-Jan-1993	Plans for the Southern Levee Restoration of East Garden Grove Wintersburg Channel, Facility No. C05, from Graham St. to Warner Ave., 13 pp.

Table 2. Reports and plans applicable to reach 1

LRC Number	Diaz Yourman GeoPentech Kinnetic Laboratories Number	Reach	Author	Type	Borings	Channel	Description	Sta	Date	Report
13	4		Bein et al.	Geotechnical evaluation		C02	Huntington Harbor outlet to Cerritos		Jun-83	Robert Bein, William Frost & Associates. Jun 1983. Project report, Bolsa Chica Channel Facility No. C02, Huntington Harbor outlet to Cerritos Avenue.
	34		Orange County Flood Control District						6-Jan-1959	As-Built Plans for the Construction of Bolsa Chica Channel, Tidelands to Cerritos Avenue, 32 pp.
	36		Orange County Flood Control District						12-Jan-1962	Plans for the Construction of Westminster Channel, McFadden Avenue to Sta. 92+20 and at Graham Street, 10 pp.
	47		Diaz · Yourman & Associates						21-Jun-1994	Geotechnical Services, Soil Sampling and Laboratory Testing, Huntington Harbour, California, 20 pp.
	51		Diaz · Yourman & Associates						4-May-2004	Initial Site Assessment, Seal Beach Regional Trail, Seal Beach, California, 188 pp.
	56		Diaz · Yourman & Associates						27-Sep-2013	Geotechnical Investigation, Bulkhead Evaluation, Sunset Harbor Maintenance Dredging, Orange County, California, 109 pp.

Table 3. Reports and plans applicable to reach 23

Risk Rating
P - Probability (1 negligible. to 5 v. likely); I - Impact (1 v. low to 5 v. high)
R - Risk, P x I (1-4 negligible, 5-9 minor, 10-14 moderate, 15-19 substantial, 20-25 severe)
Risk Control Measures (RCM)
AC - Accept; AV - Avoid; C - Contingency; M - Mitigate; T - Transfer

Table A4-1 Geotechnical Risk Register

No.	Hazard	Cause	Impact	Risk Rating			Risk Owner	Cost Impact	Schedule Impact	Risk Control Measures, RCM (including risk rating reduced and RCM type)	Risk Rating following RCM				Revised Risk Owner	Contingency Measures
				P	I	R		$$$	weeks		P	I	R	Risk Decrease		
1	**Unanticipated ground and/or subsurface behavior (subsidence) during construction**	1. Inadequate site investigation information 2. Inadequate characterization and understanding of ground conditions/geologic behavior 3. Poor construction workmanship 4. Improper selection of temporary excavation type (unsupported slope gradient, shoring type, ground improvement) 5. Contractor will take more risks than what the government accepts 6. Contractor may bid job assuming no shoring 7. Presence of peat throughout the project	1. Damage to offsite property 2. Schedule delays 3. Cost increases 4. DSC claims	3	4	12 Moderate	USACE Contractor	2%	5%	**Design Phase** - (Probability: M) Review existing subsurface information and determine need for supplemental site investigation, including test pits to expose soils and rock - (Probability: M) Develop understanding of site stability and potential triggering mechanism for ground movement - (Impact: M) Design conservatively to minimize mechanisms likely to trigger movement - (Probability: T) Prepare GBR to interpret / characterize engineering data - (Probability: M) Research previous construction excavation methods implemented in the locality and in similar ground conditions - (Probability: M) Assess impacts of dewatering on adjacent properties if active dewatering is required **Construction Phase** - (Probability: T) Require work plans from the Contractor to verify their understanding of ground conditions - (Impact: M) Inspect excavation daily (USACE construction and geotechnical staff) to ensure compliance with specifications and work plans - (Impact, M) Prepare daily field reports documenting change in ground condition - (Impact, M) Provide recommendations for monitoring and instrumentation to record ground movement - (Impact, M) Ensure adequate QA and EDC staff for control of all construction practices - (Impact, M/T) Inspect site (USACE and contractor) continuously for evidence of ground movements - (Impact, T) Conduct pre-construction survey and documentation of existing site conditions	2	2	4 Negligible	67% Moderate to Negligible	Contractor USACE	1. Stop work 2. Ensure area made stable 3. Assess site and evaluate mitigation alternatives
2	**Ability to excavate to line and grade / Feasibiltiy of rock excavation**	1. Contractor is not familiar with local geology conditions 2. Contractor does not understand or lacks experience with suitable excavation methods 3. Contractor fails to utilize appropriate equipment	1. Damage to offsite property 2. Schedule delays 3. Cost increases 4. DSC claims	4	4	16	USACE Contractor	3%	10%	**Design Phase** - (Probability: M) Research previous construction excavation methods implemented in the locality and in similar ground conditions - (Probability: M) Review existing subsurface information and determine need for supplemental site investigation, including test pits to expose soils and rock and determine excavation characteristics - (Probability: M) Delineate areas of temporary slopes and gradients by station, coordinating requirements with PDT - (Probability: M) Delineate areas of required shoring by station - (Impact: AV) Coordinate with PDT and real estate to acquire potentially impacted properties - (Probability: M/T) Research potentially suitable excavation methods (e.g. blasting) and either do not allow in specifications or transfer risk to contractor while still protecting adjacent property - (Probability: M) Provide input for typical contractor requirements, standards of practice and potential use of phasing. - (Probability: M) Evaluate if ground improvment may be needed to facilitate construction **Construction Phase** - (Probability: T) Require work plans from the Contractor to verify their understanding of ground conditions - (Probability: T) Provide GBR with specific intrepretation of geology materials and engineering properties - (Impact: A) Allow longer durations in schedule to account for contractor inefficiency so USACE can budget field support	2	3	6	63%	Contractor USACE	1. Assess site conditions for conformance with expectations 2. Require revised excavation plan with alternate methods 3. Use rock breakers/non-explosive pre-splitting of rock using expansive grouts or similar

Risk Rating
P - Probability (1 negligible. to 5 v. likely); I - Impact (1 v. low to 5 v. high)
R - Risk, P x I (1-4 negligible, 5-9 minor, 10-14 moderate, 15-19 substantial, 20-25 severe)
Risk Control Measures (RCM)
AC - Accept; AV - Avoid; C - Contingency; M - Mitigate; T - Transfer

Table A4-1 Geotechnical Risk Register

No.	Hazard	Cause	Impact	Risk Rating			Risk Owner	Cost Impact	Schedule Impact	Risk Control Measures, RCM (including risk rating reduced and RCM type)	Risk Rating following RCM				Revised Risk Owner	Contingency Measures
				P	I	R		$$$	weeks		P	I	R	Risk Decrease		
3	**Changes in quantities of geologic materials and excavation characteristics**	1. Ground conditions differing from those indicated from site investigation 2. Inadequate site investigation information 3. Inadequate characterization of engineering properties of geologic materials	1. Schedule delays 2. Cost increases 3. DSC claim	4	4	16	USACE	3%	10%	**Design Phase** - (P, T) Prepare GBR to estimate quantity of each geology material type to be encountered - (P, M) Review existing subsurface information and determine need for supplemental site investigation, including test pits to expose soils and rock and determine excavation characteristics **Construction Phase** - (I, M) Geology staff continually document and compare material being removed from excavation to expected conditions - (I, M) Report areas of unexpected ground conditions and await further instruction	2	4	8	50%	Shared	1. Document material type and quantity 2. Perform additional exploration as required to determine extent of variance 3. Compensate contractor accordingly
4	**Unexpected hard obstructions in excavations**	1. Ground conditions differing from those indicated from site investigation 2. Inadequate site investigation information 3. Poor understanding of geologic behavior	1. Schedule delays 2. Cost increases 3. DSC claim	3	3	9	USACE	1%	3%	**Design Phase** - (Probability: M) Research previous construction excavation methods implemented in the locality and in similar ground conditions - (Probability: M) Review existing subsurface information and determine need for supplemental site investigation, including test pits to expose soils and rock and determine excavation characteristics - (Probability: T) Prepare GBR to interpret / characterize engineering data **Construction Phase** - (Impact: M) Geology staff continually document and compare material being removed from excavation to expected conditions - (Impact: M) Report areas of unexpected ground conditions and await further instruction	2	3	6	33%	USACE	1. Use rock breakers/non-explosive pre-splitting of rock using expansive grouts or similar 2. Try to assess extent of variance and possible impact 2. Compensate contractor accordingly
5	**Need to pump unexpected large quantities of groundwater for dewatering**	1. Extended periods of wet weather and under-design of temporary pumping 2. Over-pumping of excavation into surface water channel. 3. Potential contamination from run-off from works 4. Improper construction methods 5. Agency restrictions on discharge	1. Schedule delays 2. Cost increases	4	5	20	Contractor USACE	5%	20%	**Design Phase** - (Probability: M) Research previous construction dewatering methods implemented in the locality and in similar ground conditions - (Probability: M) Review existing groundwater information and determine need for pump tests, pressure tests, etc. - (Probability: T) Prepare GBR/specifications to include groundwater, pumping data, and permit requirements - (Probability: M) Coordinate with oversight agency and determine/address permit requirements such as water quality standards or other discharge restrictions **Construction Phase** - (Impact: T) Require work plan from the Contractor for dewatering - (Impact: M) Inspect excavation daily (USACE construction and geotechnical staff) to ensure compliance with specifications and work plans - (Impact: AV) Allow longer durations in schedule to account for contractor inefficiency so USACE can budget field support	2	3	6	70%	Contractor	1. Enforce provisions of contractors work plan

Risk Rating
P - Probability (1 negligible. to 5 v. likely); I - Impact (1 v. low to 5 v. high)
R - Risk, P x I (1-4 negligible, 5-9 minor, 10-14 moderate, 15-19 substantial, 20-25 severe)
Risk Control Measures (RCM)
AC - Accept; AV - Avoid; C - Contingency; M - Mitigate; T - Transfer

Table A4-1 Geotechnical Risk Register

No.	Hazard	Cause	Impact	Risk Rating P	Risk Rating I	Risk Rating R	Risk Owner	Cost Impact $$$	Schedule Impact weeks	Risk Control Measures, RCM (including risk rating reduced and RCM type)	Risk Rating following RCM P	I	R	Risk Decrease	Revised Risk Owner	Contingency Measures
6	**Inability to install selected shoring type**	1. Ground conditions differing from those indicated from site investigation 2. Inadequate site investigation information 3. Improper construction practices and/or improper selection of shoring type	1. Schedule delays 2. Cost increases	4	5	20	Contractor	5%	20%	**Design Phase** - (Probability: M) Research previous construction excavation methods implemented in the locality and in similar ground conditions - (Probability: M) Review existing subsurface information and determine need for supplemental site investigation, including test pits to expose soils and rock and determine excavation characteristics - (Impact: M/T) Research potentially suitable excavation methods (e.g. blasting) and either do not allow in specifications or transfer risk to contractor while still protecting adjacent property - (Probability: M/T) Research and evaluate shoring types that have been successfully implemented in the area and for similar ground conditions. Do not allow types as directed by USACE. Supply performance standards to contractor - (Probability: M) Revise shoring design parameters based on new information or revised plans. - (Probability: AV) Consider re-alignment of culvert alignment. **Construction Phase** - (Impact: M) Prepare ECIFP and brief construction personnel on expected ground conditions - (Impact: T) Require work plan from the Contractor for shoring	2	3	6	70%	Contractor	1. Contractor to revise shoring submittal and propose another alternative
7	**Insufficient working space for contractor operations**	1. Poor assessment of contractor construction requirements and standard construction practices 2. Lack of coordination with Real Estate, property owners, local sponsor	1. Schedule delays 2. Cost increases	5	5	25	USACE	5%	20%	**Design Phase** - (Probability: M) Perform site walk and detailed survey of route to identify areas of particular concern - (Probability: AV) Determine required construction work area based on standard construction practices for entire length of excavation. Compare to expected daylight of temporary slopes. Assess economic difference between shoring and construction easements - (Impact: AV) Develop / assess construction details / methods including phasing that may successfully minimize the work area. Obtain all necessary TCE - (Probability: AV) Consider re-alignment of culvert alignment. **Construction Phase** - (Impact: AV) Develop appropriate sequencing of works - (Impact: T) Require work plan from the Contractor for excavation - (Impact: AV) Allow for extended contract schedule to accomodate phasing to reduce work area (if applicable)	1	2	2	92%	Contractor	1. Contractor to submit revised work plans
8	**Inability to economically install culverts at road / rail crossings**	1. Minimal or insufficient coordination with local sponsor 2. One design does not account for the temporary or permenant loads imposed by the other	1. Damage to offsite property 2. Schedule delays 3. Cost increases 4. DSC claims	4	4	16	USACE	0%	2%	**Design Phase** - (Impact: M) Develop design and construction details including required phasing for each road crossing - (Probability: AV) Coordinate closely between USACE, owner and local sponsor. Establish regular meetings and points of contact - (Impact: M) Geotechnical engineer to be consulted on all phasing and alternative plans - (Probability: AV) Consider re-alignment of culvert alignment.	2	2	4	75%	USACE	1. Stop work and assess
9	**Encounter unidentified utilities**	1. Responsible parties did not complete their responsibilities 2. Insufficient funding or scheduling to complete task 3. The culvert contractor is delayed or both contractors work interfer 4. Utility relocation plans are not ready, incomplete, or not coordinated with culvert design	1. Schedule delays 2. Cost increases 3. Need for additional shoring and/or evaluation of relocations 4. Additional geotechnical studies 5. Damage to utilities	5	4	20	USACE	5%	20%	**Design Phase** - (Impact: M) Develop construction details including required phasing for each utility - (Probability: M) Manage utility relocations with local sponsor funds - (Probability: AV) Coordinate closely between USACE, owner and local sponsor. Establish regular meetings and points of contact - (Impact: M) Geotechnical engineer to be consulted on all phasing and alternative plans - (Probability: M) Provide recommendations for utility relocations and design - (Probability: M) Prepare complete set of utility plans, including existing and proposed systems **Construction Phase** - (Impact: AV) Engage Underground Service Alert and/or perform potholes	2	2	4	80%	USACE	1. Stop work and assess

Risk Rating
P - Probability (1 negligible. to 5 v. likely); I - Impact (1 v. low to 5 v. high)
R - Risk, P x I (1-4 negligible, 5-9 minor, 10-14 moderate, 15-19 substantial, 20-25 severe)
Risk Control Measures (RCM)
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Table A4-1 Geotechnical Risk Register

No.	Hazard	Cause	Impact	Risk Rating			Risk Owner	Cost Impact	Schedule Impact	Risk Control Measures, RCM (including risk rating reduced and RCM type)	Risk Rating following RCM				Revised Risk Owner	Contingency Measures
				P	I	R		$$$	weeks		P	I	R	Risk Decrease		
10	**Material Placement Issues**	1. Not understanding specifications or standards 2. Remote site 3. Weather impacts on concrete placement	1. Schedule delays 2. Cost increases	5	3	15	Contractor	3%	10%	**Design Phase** - (Probability: M) Prepare clear recommendations and specifications - (Impact: M) Establish scopes of work and associated EDC and QA budgets **Construction Phase** - (Probability: M) Establish SPL QA and EDC facilities on-site managed with SPL laboratory personnel - (Impact: M) Prepare ECIFP and brief construction personnel on expected ground conditions - (Probability: T) Require applicable work plans from the Contractor such as for earthwork and concrete placement	2	3	6	60%	Contractor	1. Contractor to submit revised work plans
11	**Slope protection design measures not adequate**	1. Proposed intent not fully understood by PDT 2. Plan not vetted by engineering 3. Slope protection was not evaluated	1. Redesign needed 2. Cost increases 3. Design schedule delays	4	3	12	USACE	2%	5%	**Design Phase** - (Probability: M) Evaluate proposed ARVS for slope stability and revetment applicability - (Probability: M) Evaluate alternative slope protection measures	1	4	4	67%	USACE	1. Revise current design to include a box culvert 2. Utilize concrete or grouted stone trapezoidal channel
12	**Soil liquefaction**	1. Soils within the project have been identified as liquefiable and adjacent to the Alquist-Priolo earthquake fault zone	1. Inability of project to provide flood protection	1	17	17	USACE, Local Sponsor	5%	20%	**Design Phase** - (Probability: M) Develop a site specific seismic study and design features to address seismic impact	1	5	5	71%	USACE, Local Sponsor	
13	**Weather Impacts**	1. The rainy season corresponds with higher groundwater levels	1. Schedule delays which cost USACE money deleting project funds 2. Cost increases associated with schedule delays 3. Project funds used and local sponsor can't fund remainder of filed oversign labor halting project	5	5	25	USACE	5%	20%	**Design Phase** - (Impact: AV) Estimate conservative weather delays in specifications - (Probability: T) Require contractor to incorporate weather into dewatering and diversion control in specifications - (Probability: M) Consider work seasons in specifications - (Impact: M) Consider/evaluate use of precast culvert sections to minimixe exposure of excavation to weather	3	3	9	64%	Contractor	1. Stop work

Notice how the geotechnical risk register flows from left to right. This is the same order, and information, that would be part of a geotechnical report. Instead of specific mitigation recommendations this presents how we are going to develop those recommendations

Table A4-2 RISK CONTROL MEAURES ADDRESSING HAZARDS

		Unanticipated ground and/or subsurface behavior (subsidence) during construction	Ability to excavate to line and grade / Feasibility of rock excavation	Changes in quantities of geologic materials and excavation characteristics	Unexpected hard obstructions in excavations	**Need to pump unexpected large quantities of groundwater for dewatering**	**Inability to install selected shoring type**	**Insufficient working space for contractor operations**	Inability to economically install culverts at road / rail crossings	**Encounter unidentified utilities**	Material Placement Issues	Slope protection design measures not adequate	Soil liquefaction	**Weather Impacts**
	CURRENT	1	2	3	4	5	6	7	8	9	10	11	12	13
	Probability	3	4	4	3	4	4	5	4	5	5	4	1	5
	Impact	4	4	4	3	5	5	5	4	4	3	3	17	5
	Total Risk	12	16	16	9	20	20	25	16	20	15	12	17	25
		Moderate (Md)	Substantial (Sb)	Substantial (Sb)	Minor (Mn)	**Severe (Sv)**	**Severe (Sv)**	**Severe (Sv)**	Substantial (Sb)	**Severe (Sv)**	Substantial (Sb)	Moderate (Md)	Substantial (Sb)	**Severe (Sv)**
	Cost Imp	2%	3%	3%	1%	**5%**	**5%**	**5%**	0%	**5%**	3%	2%	**5%**	**5%**
	AFTER RISK CONTROL MEASURES													
	Probability	2	2	2	2	2	2	1	2	2	2	1	1	3
	Impact	2	3	4	3	3	3	2	2	2	3	4	5	3
	Total Risk	4	6	8	6	6	6	2	4	4	6	4	5	9
		None / Negligible (N)	Minor (Mn)	Minor (Mn)	Minor (Mn)	Minor (Mn)	Minor (Mn)	None / Negligible (N)	None / Negligible (N)	None / Negligible (N)	Minor (Mn)	None / Negligible (N)	Minor (Mn)	Minor (Mn)
	Risk Decrease	67%	63%	50%	33%	70%	70%	92%	75%	80%	60%	67%	71%	64%
	Cost Imp	0%	1%	1%	1%	1%	1%	0%	0%	0%	1%	0%	0%	1%
	Risk Control Measure													
Design	Review existing subsurface information and determine need for supplemental site investigation, including test pits to expose soils and rock and determine excavation characteristics													
	Develop understanding of site stability and potential triggering mechanism for ground movement													
	Design conservatively to minimize mechanisms likely to trigger movement													
	Prepare GBR to - interpret/characterize engineering data - estimate each quantity of geology material type to be encountered - include groundwater, pumping data, permit requirements													
	Research previous construction excavation methods implemented in the locality and in similar ground conditions													
	Assess impacts of dewatering on adjacent properties if active dewatering is required													
	Delineate areas of temporary slopes and gradients by station, coordinating requirements with PDT													
	Provide input for typical contractor requirements, standards of practice and potential use of phasing.													
	Evaluate if ground improvment may be needed to facilitate construction													
	Delineate areas of required shoring by station													
	Coordinate with PDT and real estate to acquire impacted properties													
	Research potentially suitable excavation methods (e.g. blasting) and either do not allow in specifications or transfer risk to contractor while still protecting adjacent property													
	Review existing groundwater information and determine need for pump tests, pressure tests, etc.													
	Coordinate with oversight agency and determine/address permit requirements such as water quality standards or other discharge restrictions													
	Research and evaluate shoring types that have been successfully implemented in the area and for similar ground conditions. Do not allow types as directed by USACE. Supply performance standards to contractor													
	Perform site walk and detailed survey of alignment to identify areas of particular concern													
	Revise shoring design parameters based on new information or revised plans.													
	Consider re-alignment of culvert alignment.													
	Determine required construction work area based on standard construction practices for entire length of excavation. Compare to expected daylight of temporary slopes. Assess economic difference between shoring and construction easements													
	Develop / assess construction details / methods including phasing that may successfully minimize the work area. Obtain all necessary TCE													
	Develop construction details including required phasing for each road crossing													
	Coordinate closely between USACE, owner and local sponsor. Establish regular meetings and points of contact													
	Geotechnical engineer to be consulted on all phasing and alternative plans													
	Develop construction details including required phasing for each utility													
	Manage utility relocations with local sponsor funds													
	Provide recommendations for utility relocations and design													
	Prepare complete set of utility plans, including existing and proposed systems													
	Prepare clear recommendations and specifications													
	Establish scopes of work and associated EDC and QA budgets													

Table A4-2 RISK CONTROL MEAURES ADDRESSING HAZARDS

		Unanticipated ground and/or subsurface behavior (subsidence) during construction	Ability to excavate to line and grade / Feasibility of rock excavation	Changes in quantities of geologic materials and excavation characteristics	Unexpected hard obstructions in excavations	Need to pump unexpected large quantities of groundwater for dewatering	Inability to install selected shoring type	Insufficient working space for contractor operations	Inability to economically install culverts at road / rail crossings	Encounter unidentified utilities	Material Placement Issues	Slope protection design measures not adequate	Soil liquefaction	Weather Impacts
	CURRENT	1	2	3	4	5	6	7	8	9	10	11	12	13
	Probability	3	4	4	3	4	4	5	4	5	5	4	1	5
	Impact	4	4	4	3	5	5	5	4	4	3	3	17	5
	Total Risk	12	16	16	9	20	20	25	16	20	15	12	17	25
		Moderate (Md)	Substantial (Sb)	Substantial (Sb)	Minor (Mn)	Severe (Sv)	Severe (Sv)	Severe (Sv)	Substantial (Sb)	Severe (Sv)	Substantial (Sb)	Moderate (Md)	Substantial (Sb)	Severe (Sv)
	Cost Imp	2%	3%	3%	1%	5%	5%	5%	0%	5%	3%	2%	5%	5%
	AFTER RISK CONTROL MEASURES													
	Probability	2	2	2	2	2	2	1	2	2	2	1	1	3
	Impact	2	3	4	3	3	3	2	2	2	3	4	5	3
	Total Risk	4	6	8	6	6	6	2	4	4	6	4	5	9
		None / Negligible (N)	Minor (Mn)	Minor (Mn)	Minor (Mn)	Minor (Mn)	Minor (Mn)	None / Negligible (N)	None / Negligible (N)	None / Negligible (N)	Minor (Mn)	None / Negligible (N)	Minor (Mn)	Minor (Mn)
	Risk Decrease	67%	63%	50%	33%	70%	70%	92%	75%	80%	60%	67%	71%	64%
	Cost Imp	0%	1%	1%	1%	1%	1%	0%	0%	0%	1%	0%	0%	1%
	Evaluate proposed ARVS for slope stability and revetment applicability													
	Evaluate alternative slope protection measures													
	Retain AE to evaluate current slope condition and design													
	Evaluate corrections if needed to slope													
	Estimate conservative weather delays in specifications													
	Require contractor to incorporate weather into dewatering and diversion control in specifications													
	Consider work seasons in specifications													
	Develop a site specific seismic study and design features to address seismic impact													
	Consider/evaluate use of precast culvert sections to minimize exposure of excavation to weather													
Construction	Require work plans from the Contractor: - to verify their understanding of ground conditions - earthwork (excavation, dewatering, shoring) - concrete placement													
Construction	Inspect excavation daily (USACE construction and geotechnical staff) to ensure compliance with specifications and work plans													
Construction	Prepare ECIFP and brief construction personnel on expected ground conditions													
Construction	Prepare daily field reports documenting change in ground conditions													
Construction	Provide recommendations for monitoring and instrumentation to record ground movement													
Construction	Ensure adequate QA and EDC staff for control of all construction practices													
Construction	Inspect site (USACE and contractor) continuously for evidence of ground movements													
Construction	Conduct pre-construction survey and documentation of existing site conditions													
Construction	Provide GBR with specific intrepretation of geology materials and engineering properties													
Construction	Allow longer durations in schedule to account for contractor inefficiency so USACE can budget field support													
Construction	Geology staff continually document and compare material being removed from excavation to expected conditions													
Construction	Report areas of unexpected ground conditions and await further instruction													
Construction	Develop appropriate sequencing of works													
Construction	Allow for extended contract schedule to accomodate phasing to reduce work area (if applicable)													
Construction	Engage Underground Service Alert and/or perform potholes													
Construction	Establish SPL QA and EDC facilities on site managed with SPL laboratory personnel													

Refer to ER 1110-2-1302 beginning page 15 and 26

PROBABILITY (P)		IMPACT (I)	
Description	Score	Description	Score
Very likely	5	Very high	5
Probable	4	High	4
Likely	3	Medium	3
Unlikely	2	Low	2
Very Unlikely	1	Very low	1
Not applicable	0	-	-

Factors used to Compile Risk Matrix

Probability

Consequence/Impact	5	4	3	2	1	
V Low	5	4	3	2	1	1
Low	10	8	6	4	2	2
Med	15	12	9	6	3	3
High	20	16	12	8	4	4
V High	25	20	15	10	5	5

The corners of the chart have these characteristics:
•Low impact/low probability – Risks in the upper right corner are low level, and you can often ignore them.
•Low impact/high probability – Risks in the top left corner are of moderate importance – if these things happen, you can cope with them and move on. However, you should try to reduce the likelihood that they'll occur.
•High impact/low probability – Risks in the bottom right corner are of high importance if they do occur, but they're very unlikely to happen. For these, however, you should do what you can to reduce the impact they'll have if they do occur, and you should have contingency plans in place just in case they do.
•High impact/high probability – Risks towards thebottom left corner are of critical importance. These are your top priorities, and are risks that you must pay close attention to.

To use the Risk Impact/Probability Chart follow these steps:
1.List all of the likely geotechnical/geologic hazards that your project faces. Make the list as comprehensive as possible.
2.Assess the probability of each hazard occurring, and assign it a rating. Use a scale of 1 to 5. Assign a score of 1 when a hazard is extremely unlikely to occur, and use a score of 5 when the hazard is extremely likely to occur.
3.Estimate the impact on the project if the hazard occurs. Again, do this for each and every hazard on your list. Using the 1-5 scale, assign it a 1 for little impact and a 5 for a huge, catastrophic impact.
4.Map out the ratings on the Risk Impact/Probability Chart.
5.Develop a response to each risk, according to its position in the chart. Remember, risks in the upper right corner can often be ignored, while those in the lower leftt corner need a great deal of time and attention.

Probability	(P)
Very Likely (VLk)	5
Probable (P)	4
Likely (Lk)	3
Unlikely (U)	2
Very Unlikely (VU)	1

Impact	(I)
Very High (VH)	5
High (H)	4
Medium (M)	3
Low (Lw)	2
Very Low (VLw)	1

(R)	Risk
20 – 25	Severe (Sv)
15 – 19	Substantial (Sb)
10 – 14	Moderate (Md)
5 – 9	Minor (Mn)
1 – 4	None / Negligible (N)

Hazard
Identify and describe credible events or situations that would have an impact on the achievement of goals and objectives.

Cause
Identify credible reasons that the risk event might occur. Consider credible scenarios or potential failures. Make an informed judgement of the likely inherent risk without the operation of existing controls. By considering the inherent risk we are able to prioritize and monitor that the controls that have been put in place are working as intended.

Consequences
The potential consequence or impact of the risk if it did become a project issue. A description of the potential impact on the project as a result of the risk.

Risk Owner
This is the person responsible for managing the risk and implementing each particular Risk Control Measures. Contractors, the government, and the Local Sponsor can all be risk owners.

USACE
Local Sponsor
Contractor

Avoid. Risk avoidance involves changing the project plan to eliminate the risk or to protect the project objectives (time, cost, scope, quality) from its impact. The PDT might achieve this by changing scope, adding time, or adding resources (thus relaxing this triple constraint). These changes may require higher level approval from Chief of Engineering. Some negative risks (threats) that arise early in the project can be avoided by clarifying requirements, obtaining information, improving communication, or acquiring expertise.

Transfer. Risk transference requires shifting the negative impact of a threat, along with ownership of the response, to a third party. An example would be the PDT transfers the financial impact of risk by contracting out some aspect of the work in a lump sum. Transference reduces the risk only if the contractor is more capable of taking steps to reduce the risk and does so. Risk transference nearly always involves payment of a risk premium to the party taking on the risk. Transference tools can be quite diverse and include, but are not limited to the use of: warranties, guarantees, incentive/disincentive clauses, different contract vahicles (would need to talk to contracting for options).

Mitigate. Risk mitigation implies a reduction in the probability and/or impact of an adverse risk event to an acceptable threshold. Taking early action to reduce the probability and/or impact of a risk is often more effective than trying to repair the damage after the risk has occurred. Risk mitigation may take resources or time and hence may represent a tradeoff of one objective for another. However, it may still be preferable to going forward with an unmitigated risk. The key is to recognize the risk up front.

Accept. This may be appropriate when consequences are not severe. Acceptance does not necessarily correlate to a lack of action. A response plan can be prepared and kept in hand, should the risk event occur.

Examples of Hazards
Landslides, expansive soils, corrosivity, settlement, seismicity (liquefaction, etc), differing site conditions, weak/compressible soils, slope stability, foundation construction, subsidence, groundwater, contamination, unforeseen ground conditions, inadequate geotechnical investigation, inappropriate design, sinkholes, rockfall, rippability, temporary excavations, dewatering, weather, underground obstructions

To successfully implement a project, you must identify and focus your attention on middle and high-priority risks – otherwise you risk spreading your efforts too thinly, and you'll waste resources on unnecessary risk management.

With the Risk Impact/Probability Chart, you map out each risk – and its position determines its priority. High-probability/high-impact risks are the most critical, and you should put a great deal of effort into managing these. The low-probability/high-impact risks and high-probability/low-impact risks are next in priority, though you may want to adopt different strategies for each.

Low-probability/low-impact risks can often be ignored.

For each Risk Control Measure (RCM) state specifically how you are controling the risk (avoid, transfer, mitigate, contingency) and what you are controlling, the Probability and/or the Impact.

Dealing With High-Impact, Low-Probability Risks

High-impact, low-probability events in general cannot be covered by contingencies. In these cases, the computation of the expected loss for an event as the product of the cost if the event occurs times the probability of the event is largely meaningless. As an extreme example, suppose an estimate for excavation is expected to cost $1,000,000 if the excavation is in soil and $50,000,000 if it encounters rock. One would certainly not assign a contingency of $49,000,000 to a $1,000,000 project. If the probability of the event is estimated as 0.02, the expected loss due to the risk event is $1,000,000. Cost engineering would not assign this number as a contingency because the estimated cost with contingency would rise 100 percent to $2,000,000. If the event occurs, the contingency of $1,000,000 will be completely inadequate to cover it, with a shortfall of $49,000,000. If the event never occurs, the additional $1,000,000 is likely to be spent anyway, so that the net effect is simply to double the cost of the project. We would soon be out of work.

High-impact, low-probability events must be mitigated by reducing the impact or the likelihood, or both. But risk mitigation and management are not cost-free. In the example above, it might be worth it to expend as much as $1,000,000 more to mitigate the $50,000,000 risk, and perhaps more than $1,000,000 if the project could be shut down mid-construction. It is difficult to seek more money in the middle of construction. In determining the budget allocation needed to mitigate high-impact, low-likelihood risks, it is necessary to identify specific risk mitigation activities. These activities should then be included in the project budget and schedule.

Risk Transfer

There is a common adage about risk management that risks should be allocatted to the parties best able to manage them.
This is far easier said than done. It is impossible to assign risks when there is no quantitative measurement of them. Risk allocation without quantitative risk assessment can lead to attempts by all project participants to shift the responsibility for risks to others, instead of searching for an optimal allocation based on mutually recognized risks. Contractors generally agree to take risks only in exchange for adequate compensation. To determine a fair and equitable price that the owner should pay a contractor to bear the risks associated with specific uncertainties, it is necessary to quantify the risks.

You need to explicitly identify all project risks to be allocated to the contractors and to the government, and these risks should be made known to prospective bidders. In order to use low bid approach to contracting, and to avoid surprises and requests for equitable adjustment, it is necessary that all parties have full knowledge of the magnitude of the risks and who is to bear them.

Risk transfer can be entirely appropriate when both sides fully understand the risks compared to the rewards. The party that assumes the risk does so because it has knowledge, skills, or other attributes that will reduce the risk. It is then equitable and economically efficient to transfer the risks, as each party believes itself to be better off after the exchange than before and the net project value is increased by the risk transfer.

Risk Acceptance

Risk Contingencies

Use monetary contengincies (or risk hedging) is the establishment of some reserve or buffer that can absorb the effects of many risks without jeopardizing the project. A large contingency reduces the risk of the project running out of money before the project is complete. This can also include the allocation of additional time, manpower, machines, or other resources used by the project. It can mean oversizing equipment to allow for uncertainties in future requirements.

Contingencies are often applied by project contractors as well as the government. Overestimating project quantities, man-hours, or other costs is used by many project participants. If jobs are awarded on the basis of lump-sum, fixed-price bids, then too much contingency can be detrimental to contractors' ability to compete. Contractors and sub-contractors may compensate by overestimating project or activity durations. Schedule contingencies allow contractors to adjust their workforce and resource allocations within projects and across multiple projects.

Cost or schedule overestimates and other factors can accumulate across a project and can be to the government's detriment because they can easily result in a general upward trend in the expected project costs and durations. In private projects, this trend is controlled by competitive factors and by the owners' knowledge of what costs and schedules should be. However, in government work and if the bidding pool is small, or if the owner is not knowledgeable, there may be inadequate controls on scope creep, cost creep, and schedule creep.

Risk Avoidance

Risk avoidance is the elimination or avoidance of some risk changing the parameters of the project. It seeks to reconfigure the project such that the risk in question disappears or is reduced to an acceptable value. The nature of the solution may be engineering, technical, financial, political, or whatever else addresses the cause of the risk. However, care should be taken so that avoiding one known risk does not lead to taking on unknown risks of even greater consequence.

Risk avoidance is an area in which quantitative, even if approximate, risk assessments are needed. For example, the project designers may have chosen solution A over alternative B because the cost of A is estimated to be less than the cost of B on a deterministic, single-point basis. However, quantitative risk analysis might show that A is much riskier than the alternative approach B. The function of quantitative risk assessment is to determine if the predicted reduction in risk by changing from alternative A to alternative B is worth the cost differential.

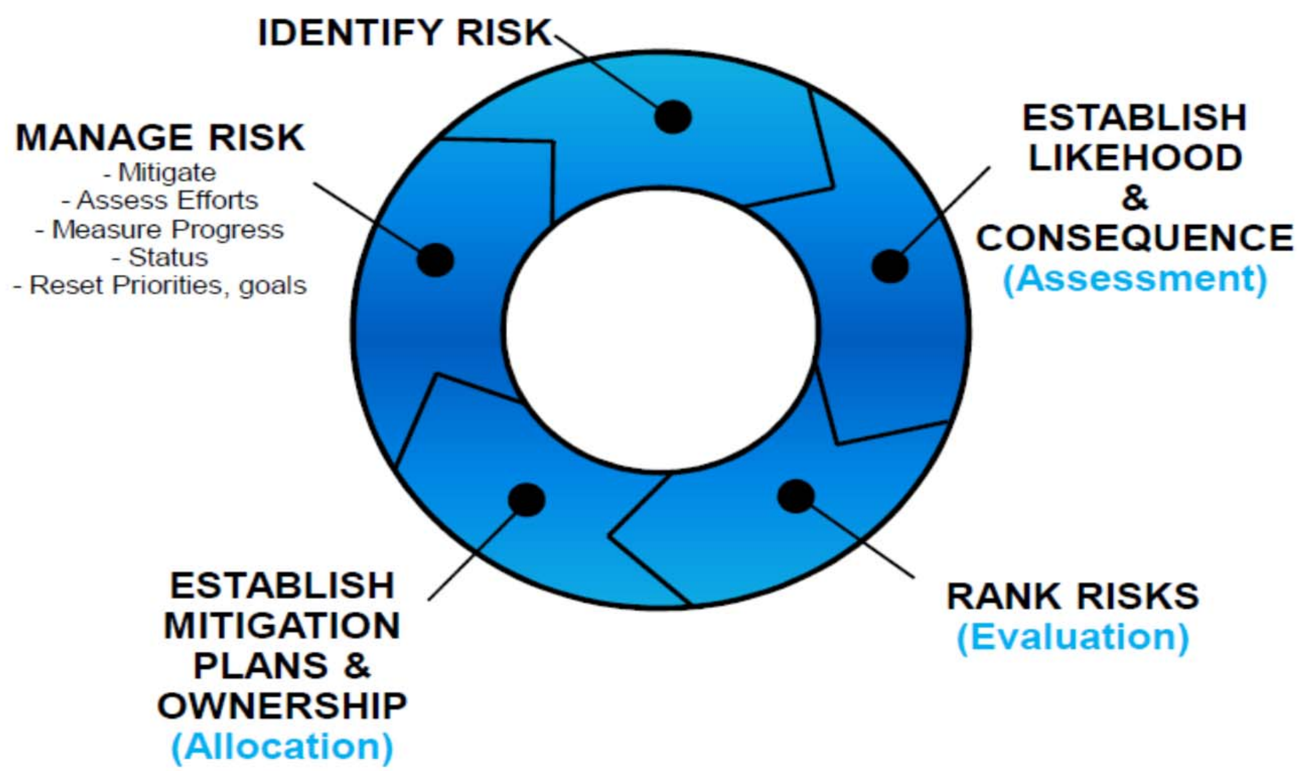
Risk avoidance is probably underutilized as a strategy for risk mitigation, whereas risk transfer is overutilized—owners are more likely to think first of how they can pass the risk to someone else rather than how they can restructure the project to avoid the risk. Nevertheless, risk avoidance is a strategy that can be employed by knowledgeable owners to their advantage.

Risk Mitigation

Risk mitigation refers to assuming a risk but taking steps to reduce, mitigate, or otherwise manage its impact or likelihood. Risk mitigation can take the form of installing excavation monitoring systems that provide information to assess more accurately the impact, likelihood, or timing of a risk. If warning of a risk can be obtained early enough to take action against it, then information gathering may be preferable to more tangible and possibly more expensive actions.

Risk mitigation, like risk avoidance, is not necessarily inexpensive. If the project is short on S&A, EDC, and QA funding, presenting a risk, then one solution might be to plan to accelerate the project, even at some considerable cost, to reduce project management risk by working 6 days a week, 10 hours a day; this is a typical strategy in privately funded projects. An example of a risk mitigation method is to conduct additional field exploration to assess anticipated subsurface conditions and excavation characteristics in detail.

Risk Management Process



Risk Identification

Risk Assessment

LIKELIHOOD	DESCRIPTION OF FREQUENCY OF EVENT	PROBABILITY	SCALE VALUE
Almost Certain	Event occurs many times during period of project or single event has high likelihood of occurrence	>70%	5
Often	Event occurs several times during period of project or single event has moderate likelihood of occurrence	40 – 70%	4
Likely	Event could occur during period of project	20 – 40%	3
Possible	Event is unlikely to occur, but it is possible during period of project	10 – 20%	2
Rare	Event is so unlikely that it can be assumed not to occur during period of project	0 – 10%	1

Risk Assessment

CONSEQUENCE	DESCRIPTION OF EFFECT OF EVENT				SCALE VALUE
	COST (IN MILLIONS)	SCHEDULE	SAFETY	PROJECT PERCEPTION/ POLITICAL REACTION	
Catastrophic	Adds up to $250	Adds 12 months	Multiple public accidents	Public perception very poor. Project seriously jeopardized. Serious political consequence to -- Owner	5
Major	Adds up to $100	Adds 6 months	Single public accident and multiple workforce accidents	Project jeopardized. Requires considerable effort to regroup public/political support	4
Moderate	Adds up to $50	Adds 4 months	Single public accident or multiple workforce accidents	Some concern for project viability. Some political consequence experienced by Owner. Moderate effort required to re-establish viability.	3
Minor	Adds up to $25	Adds 2 months	Single workforce accident	Minor concern for project viability and effect on Owner politically	2

	of project.		

Insignificant	Adds up to $10	Adds 1 month	Little possibility of accident	Little or no concern for project viability and effect on Owner politically	1

Traditional Practice of Risk Ranking

CONSEQUENCE PROBABILITY	INSIGNIFICANT (1)	MINOR (2)	MODERATE (3)	MAJOR (4)	CATASTROPHIC (5)
RARE (1)	1	2	3	4	5
POSSIBLE (2)	2	4	6	8	10
LIKELY (3)	3	6	9	12	15
OFTEN (4)	4	8	12	16	20
ALMOST CERTAIN (5)	5	10	15	20	25

Risk Allocation

- Mitigate
- Transfer (e.g.: Share, Insurance)
- Accept
- Avoid

Identify party best able to implement the allocation

Table A4-3 Cost and Schedule Impact Data Base for Development of Consequences

Project Name	Project Features	Contract Price and Schedule	Project Difficulty	Contract Modification	Describe the Modification Reason and Specific Hazard/Event/Situation	Schedule Impacts	Describe damages to off-site property, if any	Was Excavation Monitored and Instrumentated	Was Sufficient EDC/QA Performed (If not, why not)?	Lessons Learned
Clay Avenue Wash Detention Basin	Concrete Spillway, outlet works, retaining walls, dam embankments, utilities, penetrations, riprap	Award- $5.1million	medium	Total- $6.4million Mostly pertaining to reconstruction claims Does not account for complete reconstruction costs	No EDC and QA. Contractor placed defective embankment fill, constructed dam in wrong footprint, left construction debris and forms in dam, defective practices	Years	Unk	No	No. Poor communication and funding limitations.	EDC and QA are requirements, not luxuries.
Tucson Drainage- Basins 1, 2, 3, and 4	Basins, outlet works, dam embankments, concrete channels, soil nail walls	Award- $19.4	medium	Total- $20.4million	Trash being left in place under dam embankment, substandard to no QA, erosion damage to slopes during rains, trash left on basin slopes			No	No	
Nogales/Chula Vista Flood Control Project	Sewer line replacement, channel deepening and widening, grouted stone, bridge replacement	Award- $5.8million	medium	Total- $12.2million (110%) Sewer pipe- $1million; delay claims- $1million; design changes remainder	Poorly constructed temporary shoring and excavations, poor dewatering program, soft and weak foundation soils	482 days	Damages to residences during dewatering	No	No	
Nogales/Chula Vista Flood Control Project Bridge Replacement	Bridge replacement, CIDH piles, limited approach channel work	Award- $4million	medium	Total- $4.6million (15%) DSC Claim; all design changes	Contractor has a DSC claim. Encountered debris during CIDH pile construction.		none	Yes	Yes	
Reach 9 Phase 2A	Sheet Pile wall; grouted stone (engineered slopes and not engineered); grouted stone key into bedrock; soil nail wall	Award- $19.9million	high	Total-$26million (31%) Overdrill $280k; Sheetpile redesign $3.8million Soil Nail Wall $200k; Grouted stone key $280k	Pile Driving- left the word "refusal" in the specifications	570 days	none	yes	yes	Reach 9 Phase 2A LESSONS LEARNED: NUMBER 1-Clear and Grub 1 year prior to advertise and perform thorough field investigation!!! 1. SPECIFICATIONS
Reach 9 Phase 2B	Levee, grouted stone, dewatering and diversion, retaining walls, tie-back walls	Award- $16.6million	medium	Total- $38.4million (140%) Utilities- $1million; Retaining walls- $1million Environmental- $6million; stone and grout- $2million	Obvious design deficiencies with retaining wall additions. Large dollar changes were environmental. Stone and grout quantities.	1700 days	Unk	Unk	Unk	Better design preparation and review.
Reach 9 Phase 3	Soil cement river bank protection	Award-$4.8million	medium	Total-$5.3million (10%) No mods pertaining to geotechnical hazards	Ground cracking during excavation No mods pertaining to geotechnical hazards		None	Yes	Yes	Monitoring of instrumentation and daily site visits reduced impacts
Reach 9 Phase 4	Soil cement river bank protection	Award- $13.3million	medium							
Reach 9 Phase 5A	Sheetpile, grouted stone revetment, protection of off-ste property	Award- $20.3million	medium		Dewatering effluent, plan of excavation			Yes	Yes	
Reach 9 Phase 5B	Grouted stone revetment	Award- $25.4million	medium	Additional Asphalt Removal - $19,125.12 Toe Down - Not negotiated yet Option B - Not negotiated yet	Dewatering effluent, plan of excavation, toe down design change toe down depth	Toe Down - Not negotiated yet Option B - Not negotiated yet	None	Yes	Yes	
Murrieta Creek Phase 1			medium		extensive channel repairs as a result of early event.					
Murrieta Creek Phase 2	Flood control channel, Riprap, Turfmat, soil cement, shoring	Award- $16million	medium	Rev Qty Soil Cement - $481,000 RFP # 0007- Credit for Shoring & Proposal for Additional Main Street Shoring RFP # 0018-Project Drainage REA # 10: Soil Cement and Structural Backfill Quantity Increase	Ran out of EDC funds Contractor struggled with project Bad topography	4 Weather Delays - 146		yes, by contractor with problems	Yes	
Adobe Dike	penetrations, utility relocation, buttresses, MSE wall, surcharge	Award- $6.4million	medium	Total- $9.4million (47%) Quantity adjustments- $1million	Utility coordination by local sponsor Geotechnical keyways deleted from final plans prior to award	700 days	None	Yes	Yes	
CIW Dike	Riprap, zoned embankment, spillway, surcharge, interior drainage	Award- $12.7million	medium	Total-$17.4million (37%) North Dike $3.4million DSC 139 days Geotechnical other $500k	Encountered Differing Site Conditions grout the stone adjacent to a channel Bad riprap gradation	139 days	None	Yes	Yes	Note that impacts were limited by performance of EDC/QA.
Prado Main Embankment and Outlet Works	Dam raise, gated outlet works, outlet channel, cofferdam	Award- $67.4million	high	Total-$98.1million (46%) Design changes gradations-$1million; cofferdam $1million; MSE Walls-$1million Cement admixtures-$2million; a lot of <$1million chages for design defects; Utilities- $2million	There were 7 pages of modifications. The majority of these involved design changes and quantity increases.	There are over 2,000 days recorded. Not sure if they overlap	Unk	Unk	Unk	
Prado Auxiliary Dike and Floodwall	Dam embankment extension, floodwall, interior drainage, riprap	Award- $13.8million	medium	Total- $14.1million (minimal)	No significant geotechnical related modifications		Unk	Unk	Yes	
Prado Housing and Sewage Dikes	Soil cement, earth embankment, interior drainage, spillways, riprap	Award- $13.2million	low	Total- $15million (36%) Utility relocation- $2million Design defect reconstruction- $2.7million	Mods offset by option deletions Design defects and poor coordination	350 days	Unk	No	No	DSAC II rating. Had to mitigate with separate construction contracts. Design and oversight insufficient.
San Luis Rey River Levees	Grouted stone, levees		medium		Loss of erosion protection, loss of ground at top of levee, loss of level of protection Failure of relief wells		None	No	No	Can't issues LSER without full geotechnical study. Will cost millions to mitigate omissions. There is no documentation regarding construction.

Project Name	Project Features	Contract Price and Schedule	Project Difficulty	Contract Modification	Describe the Modification Reason and Specific Hazard/Event/Situation	Schedule Impacts	Describe damages to off-site property, if any	Was Excavation Monitored and Instrumentated	Was Sufficient EDC/QA Performed (If not, why not)?	Lessons Learned
Calexico BP Pavements	Road and parking lot asphalt pavement		low		Added geogrid, filter fabric, and gravel to subgrade. Very soft subgrades were found; much softer than design had anticipated. Asphalt pavement would have been immediately damaged upon placement if subgrade had not been stabilized		no impact	No	Yes	Do more exhaustive investigation for pavement design including groundwater observations and multiple DCP soundings.
Sweetwater River Flood Control Channl	Channel widening and grouted stone slope revetment		low		Repair voids behind grouted stone formed by seepage. Excavate, place filter fabric and gravel for subdrains with outlet pipes, backfill, replace grouted stone. Seepage from out of bank behind revetment caused voids which were growing. Repair necessary to preserve integrity of revetment.		no impact	No	No	provide better back-drainage for grouted stone revetments; provide filtering on the backdrains to prevent piping erosion.

Table A4 - 4 Summary of Hazard vs Consequences

Hazard	Cost Impact	Percent of Contract Price	Schedule Impact / Percent of Original	Projects in Data Base
Difffering Site Conditions (DSC)	$3.7m $0.6m			CIW Nogales
Inadequate Site Investigation	$2M to 4.3M (not easily quantified; a portion of this amount would be included in	Percent of award price. 10% to 20%.	365 days	Reach 9 Phase 2A
Inadequate Site Investigation				Calexico BP Pavements
Ground Deformation				Reach 9 Phase 3
Poor Construction				CAWDB, San Luis Rey, Tucson Drainage, Nogales
Utilities				Adobe Dike
Inappropriate Design				Sweetwater River Flood Control Channl
Flood Protection level and season for construction				Prado cofferdam Murrieta Creek Phase 1
weather				

Appendix G5 Westminster Borings Logs Compiled from As-Built Drawings and Geotechnical Reports

Overview

This section presents a summary of the borings advanced in and around the proposed project. Cross sections, methodology for estimating soil parameters, and general geotechnical descriptions of the soils in the channels area provided.

The borings available from as-built drawings and geotechnical reports in the area are summarized in Figure 1. Boring locations were either located based on surveyed location provided on the boring log, or georeferenced from as-built drawings. Cross sections were then cut for subreaches containing soil borings as shown in the attached. The ground elevation was determined by available LIDAR data for the bottom of the channel centerline.

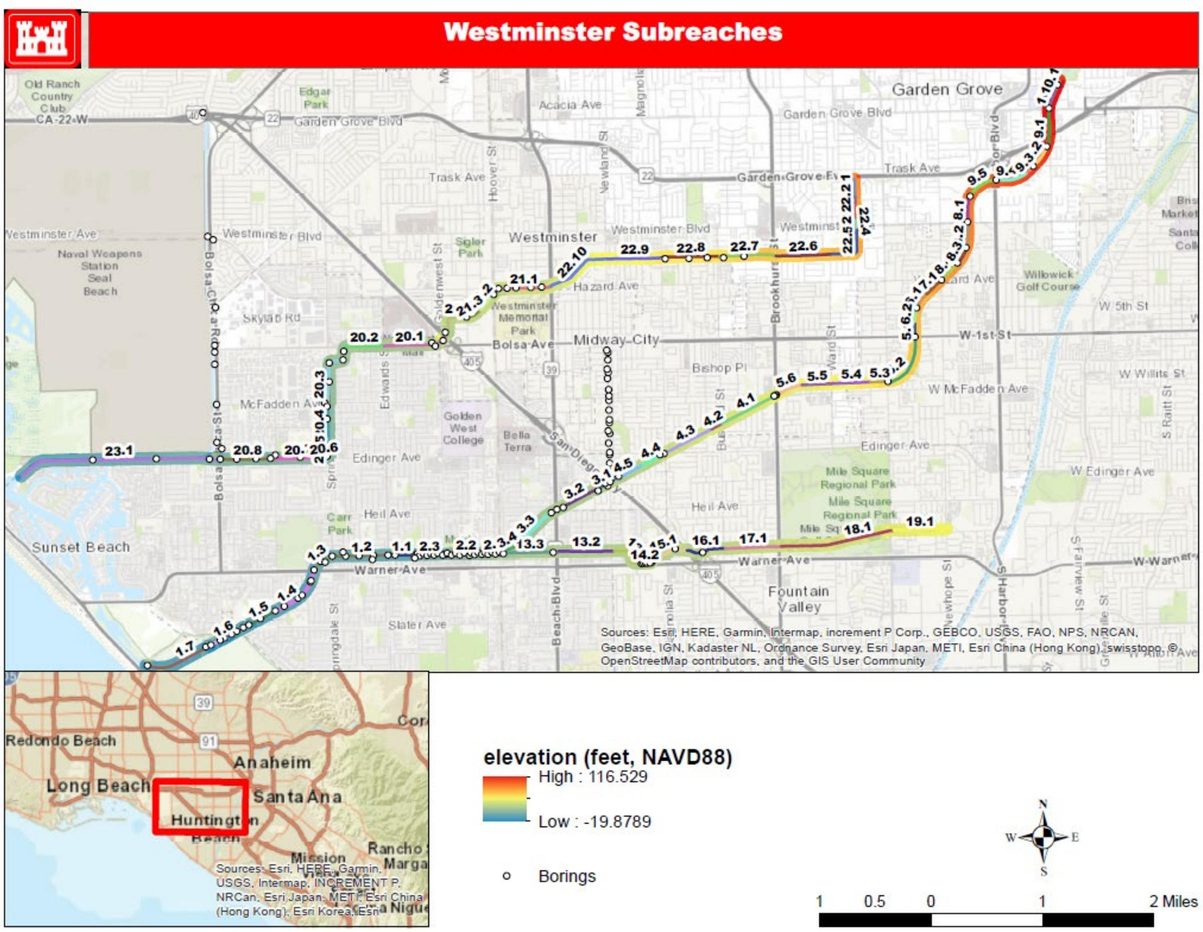


Figure 1. Borings by subreach (see attached for full scale)

Estimation of Soil Properties

The soil properties were estimated based on Unified Soil Classification System (USCS) descriptions and available blow counts. For clayey soils, the cohesion was estimated based on blow counts using a correlation developed for Chicago Soils as shown in Figure 2.

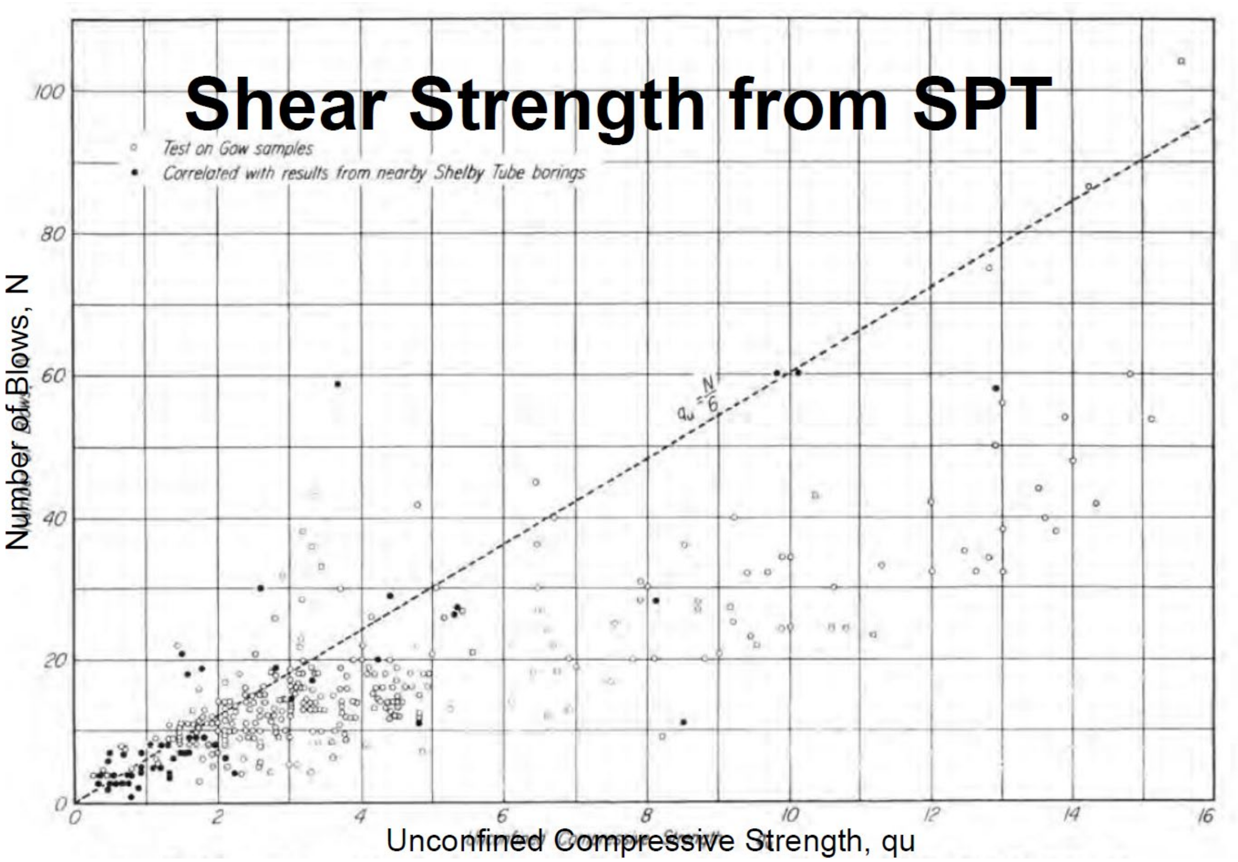


Figure 2. Shear strength from SPT blow counts (Peck and Reed, 1955)

The shear strength (c) can then be calculated as follows:

$$q_u = \frac{N}{6}\left(\frac{tons}{ft^2}\right)$$

$$c = \frac{q_u}{2} * \frac{2000\ lb}{1\ ton} = q_u * 1000 = \frac{N}{6} * 1000\ (\frac{lb}{ft^2})$$

Clay consistency can be estimated based on blow counts as shown in Figure 3. For low plasticity clays with soft to very soft consistency, the natural water content was estimated to be 40%, or roughly equivalent to the liquid limit of a low plasticity soils. For medium to stiff soils, the natural water content was estimated to be 30%. For stiff to hard low plasticity clays the natural water content was estimated to be 20%. Assuming a specific gravity of solids of 2.7, the unit weight of soils is estimated as shown in Table 1.

Sample Description

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT, additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits is estimated based on visual observation and is presented parenthetically on the test pit logs.

Density SAND or GRAVEL	Standard Penetration Resistance (N) in Blows/Foot	SILT or CLAY Consistency	Standard Penetration Resistance (N) in Blows/Foot	Approximate Shear Strength in TSF
Very loose	0 - 4	Very soft	0 - 2	<0.125
Loose	4 - 10	Soft	2 - 4	0.125 - 0.25
Medium dense	10 - 30	Medium stiff	4 - 8	0.25 - 0.5
Dense	30 - 50	Stiff	8 - 15	0.5 - 1.0
Very dense	>50	Very stiff	15 - 30	1.0 - 2.0
		Hard	>30	>2.0

Moisture

Dry	Little perceptible moisture
Damp	Some perceptible moisture, probably below optimum
Moist	Probably near optimum moisture content
Wet	Much perceptible moisture, probably above optimum

Minor Constituents	Estimated Percentage
Not identified in description	0 - 5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

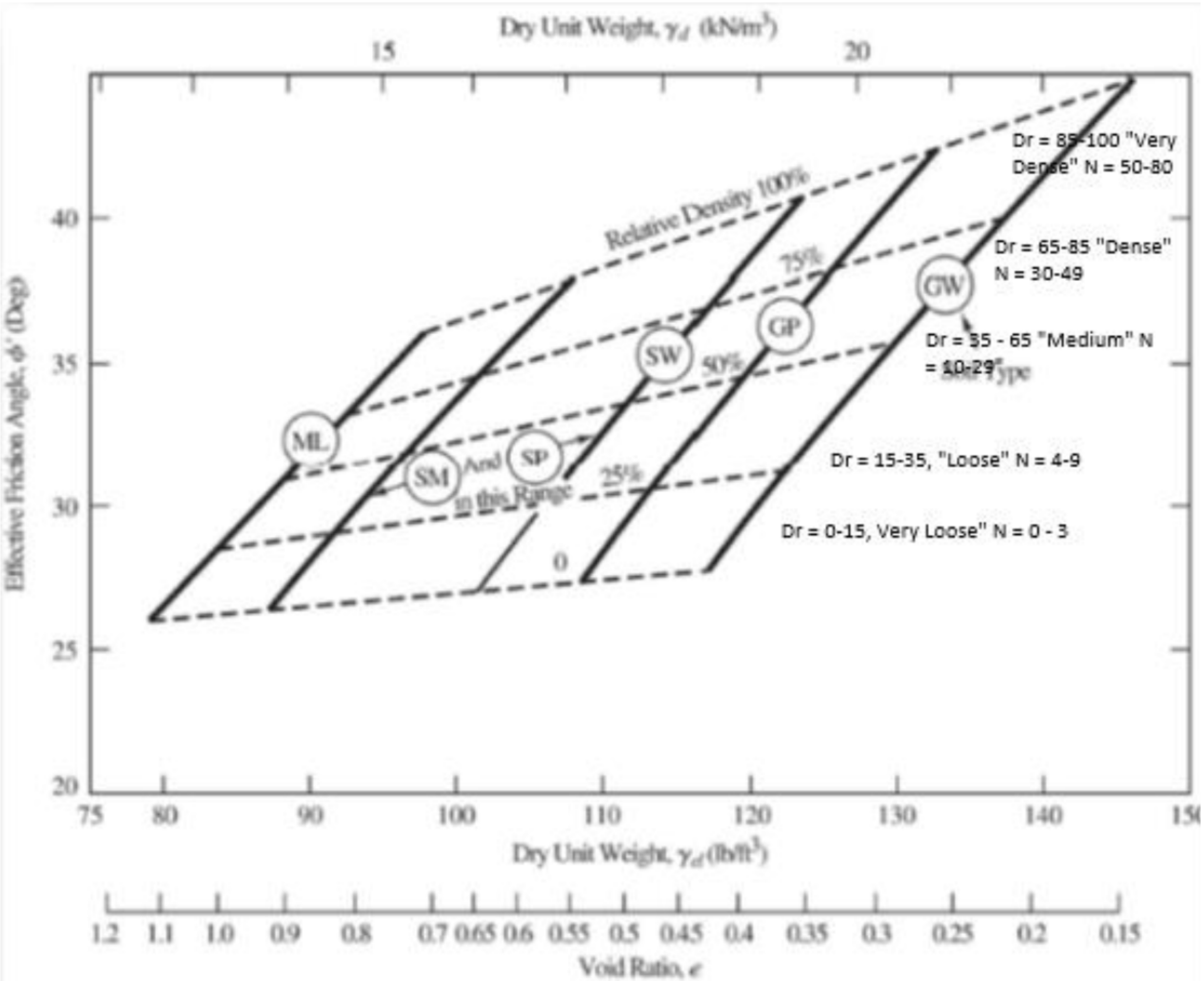
Figure 3. Soil consistency based on blow counts

Consistency	SPT blows per foot (N)	γ (pcf)
Very soft to soft	0 – 4	120
Medium stiff to stiff	4 - 15	125
Very stiff to hard	>15	130

Table 1. Estimation of unit weights of low plasticity soils based on blow counts

For granular soils (SP, SM, SW, GP, GM, GW, etc.) the dry unit weight and friction angle was estimated based on blow counts as shown in Figure 4. Granular soils were assumed to be saturated when estimating unit weight and moist unit weight was assumed to be the same as saturated unit weight. Saturated and moist unit weight is then calculated as follows

$$\gamma_{sat} = \gamma_m = \gamma_w - \frac{\gamma_d}{G_s} + \gamma_d$$



Description of Cross Sections

What follows is a description of the cross sections developed for the sub-reaches where data were available. Often blow counts were available in addition to USCS descriptions. Additionally borings were often available at regular intervals (less than 1000 ft) which permitted development of cross sections. To accurately develop these cross sections the blow counts were needed in addition to the USCS designations because often a hard/dense layer was discernable. This means that there were hard or dense layers that would affect geotechnical or structural design at the bottom of the borings that appear in multiple borings indicating that this is a consistent geologic layer.

Channel C02

Channel C02, which is the shortest channel and extends from the channel which ultimately exits in the Anaheim bay east to Bolsa Chica Road for this project. The levees in this area appear to be silt and silty sand, which can be erodible. The borings available in this area are missing blow counts, which would help characterize the strength and stratigraphic layers.

Subreach 23.1

Subreach 23.1 consists of a layer of silty clay and silt that ranges between 6 and zero feet, which overlies a 2 ft thick sand layer, overlaying a silty clay layer of zero to 3 feet at the channel centerline. Above the channel bottom there are silts, clays sands, silty clays, organic silts, and poorly graded sands. There were no available blow counts. As no structures are proposed in this area soil strength data may not be required for this subreach.

Channel C04

Channel C04 begins at Bolsa Chica Road and extends east and north as shown in Figure 1. Channel C04 consists of mainly inorganic soils. The main concern for this channel is lack of data. No geotechnical data were found for subreaches 22.10, 22.9 and 22.6 – 22.1. Additionally, the levees in this area appear to be silt and silty sand, which can be erodible.

Subreach 20.8 to 20.6

Subreach 20.8 to 20.6 has a thick layer of medium stiff to stiff low plasticity clay and silt to a depth of almost 40 ft, which thins to the east. At approximately 4000 ft from the west end of C04 the clay thins to approximately 10 ft and in underlain with a dense sand. There is a layer of high plasticity clay that appears in two borings in this area below the medium stiff to stiff clay.

Subreach 20.5 – 20.3

From subreach 20.5 through 20.3 there is a layer of medium stiff silt and low plasticity clay that is at least 20 ft thick.

Subreach 20.2 – 20.1

There are no available borings from subreach 20.2 – 20.1

Subreach 21.4 – 21.1

Subreach 21.4 to 21.1 consist mainly of medium stiff low plasticity clay, silt, and silty sand with a couple lenses of high plasticity clay.

Subreach 22.10 – 22.9

There are no available borings from subreach 22.10 – 22.9

Subreach 22.8 – 22.8

Add description

Subreach 22.6 – 22.1

There are no available borings from subreach 22.10 – 22.9

Channel C05

Channel C05 is the most geologically variable of the channels. It begins with a complex system of sand dunes, high plasticity clay, peat, and organic silt zones on the west end and becomes more homogenous and inorganic upstream beginning in subreach 4.5. Subreach 4.5 starts just east of the San Diego Freeway. Subreach 7.1 to the end at subreach 10.1 consists mainly of medium dense to dense poorly graded sands, and silty sands. Though there are numerous borings on the west end of Channel C05, there are a few subreaches without borings and many that are missing borings to adequately characterize the geology.

Subreach 1.7

Subreach 1.7 begins with clayey sand at the surface and transitions to thirty feet of sand on the west end of this subreach, which is approximately 3000 ft from the west end). The sand is dense, but the clayey sand is soft.

Subreach 1.6

Subreach 1.6 starts with hard silty clay and clayey sand and thins to the east. Under this layer is a 10 to 20 ft thick layer of medium dense sand, which is underlain by some pockets of hard clay. Within the hard clay layers is a layer of medium stiff silty clay which is then underlain by hard clay and dense sand

Subreach 1.5

Subreach 1.5 is similar to 1.6 with a thin layer of fine grained material (silty sand transitioning to silty clay) which is underlain by a medium dense sand, then a stiff silty clay. Under the silty clay is a layer of medium dense silty sand which transitions to dense sand to the east. There is a layer of well graded sand under the medium dense silty sand. A pocket of poorly graded gravel was found in B-02-12. At the bottom, roughly 50 ft below the bottom of the channel is a layer of dense sand.

Subreach 1.4

Subreach 1.4 has surface soils (5- 7 ft) of silt and poorly graded sands underlain by approximately 20 ft of silty clay to silty sand. A 15 ft thick layer of peat was found in boring B-04-1A which is located a little more than a third of the way from west to east. However peat was not found in other borings in this subreach. Under the fine grain materials is a layer of poorly graded sand. However blow counts for this sand were not available.

Subreach 1.3

Subreach 1.3 has 20 to 30 ft of organic silt and peat at the surface. There is a beach ridge approximately 2/3 the distance from the west of the subreach. Under the peat and organic silt is a layer of poorly graded sand. In the beach ridge area there is also interbedded high plasticity clay, peat, well graded gravel and sand. Below these layers is a dense sand from approximately -40 ft to -60 ft.

Subreach 1.2

Subreach 1.2 has a layer of loose silt and sand on the west end. There are two deposits of organic silt that extend 30 ft deep from the surface that are separated by what appears to be a beach ridge of medium dense poorly graded sand.

Subreach 1.1

There is a shallow deposit of peat to the west end of this reach. The rest of the reach is underlain by a medium dense sand to approximately – 40 ft. Then there is a layer of dense poorly graded then well graded sand. There are no borings in the middle to west end of this subreach.

Subreach 2.3

There is approximately 30 ft of organic silt on the west end of subreach 2.3 and a 20 ft thick layer of interbedded organic silt, sand, peat, high plasticity clay and silt on the east end. Between these two orgainic layers is an area of medium dense silt. Starting at a depth of approximately 20 ft the soils become harder and denser. About ¾ of the section on the west side is underlain by stiff to hard silts and clay from -20 to -40 ft. The remaining ¼ to the east is underlain by dense sands from -20 to -40 ft. Then the bottom of the subreach (-40 to -60) appears to be underlain by dense sands.

Subreach 2.2

The surface down approximately 10 ft for approximately the western 2/3 of the subreach consist of peats and interbedded peat, high plasticity clay silt and peat. The eastern 1/3 consists of medium dense to dense poorly graded sand. Below these layers from approximate elevation -10 to -30 there are stiff silts and clays. Below the silts and clays there are hard silts and clays interbedded with sand.

Subreach 2.1

Subreach 2.1 consists of stiff clays for the top 10 ft. There are no deep borings to the west end of the subreach. On the east end there are layers of high plasticity clay, silty clay, silt, low plasticity clay, silty sand, poorly graded sand and silt. The lower layer of sand is dense and the silt is stiff to hard.

Subreach 3.3

There is only one boring for subreach 3.3 and it appears to be offset from the centerline. There is organic silt for the top foot and then poorly graded sand for the next 2-3 ft. This subreach is missing borings for the west and center portion.

Subreach 3.2

Subreach 3.2 has a layer of organic silt on the west end that is nearly 10 ft deep. There is a ridge of silt and poorly graded sand to east of the organic silt. From a depth of 10 to nearly 20 ft there is a layer of stiff silty clay. This subreach is missing borings in the center and eastern portions.

Subreach 4.5

This subreach consists of poorly graded sand for the top 7 ft from the bottom of the channel. The boring was likely performed during the construction of I-405 and is offset from the channel alignment. There are no borings to the middle or eastern portion of this subreach.

Subreach 4.4

Subreach 4.4 consists of approximately 6 ft of silty clay underlain by 8 ft of poorly graded sand. The silty clay is stiff. This subreach is missing borings on the west and center portions.

Subreach 4.1

This subreach consists of medium dense silty sands, silts, some medium to stiff clay. From an elevation of 5 to -10 ft (bottom of boring) the sand and silty sand is dense.

Subreach 5.3

The top 12 ft or so consist of very stiff to stiff silty clay. The bottom 6 ft consist of hard silty clay. There are no borings

Subreach 5.2

No borings for this subreach.

Subreach 5.1

To the east end of this subreach there is a boring showing 5 – 6 feet of loose sand which overlays 12 feet of stiff to very stiff silty clay. There are no borings to west and center portions of this subreach.

Subreach 6.2

No borings for this subreach.

Subreach 6.1

There is one boring toward the western side of this subreach that consists of poorly graded sand for the top 2 – 3 ft, then 2 feet of stiff silty clay, which overlies 12 feet of dense poorly graded sand.

Subreach 7.1 to 10.1

This subreach consists mainly of poorly graded sand. Borings in this area extend approximately 10 ft below the bottom of the channel. The sand ranges from medium dense to dense.

C06

Channel C06 begins at the confluence of Channel C05 and C06 just east of Gothard and a rail line. Channel C06 has only a few borings with many of the borings lacking blow counts that would help characterize the soil strength and stratigraphy.

Subreach 13.3 – 13.1

There are two borings to west end of this subreach. The western most borings stiff to very stiff silty clay for the top 10 feet underlain by medium dense well graded sand. The boring approximately 1/3 from the western end has an approximately 7 ft thick layer of organic silt underlain by silty clay. This subreach is missing borings to the center and eastern portion.

Subreach 14.2 to 16.1

There were a number of borings to the western portion of these subreaches that show silt and silty clay in the leveed area and then clay and silty sand in the area beneath the channel bottom. There is one boring in the central portion of this subreach that shows silt. The boring in the center shows the silt to be very loose. However there are no blow counts for the borings to the west of this subreach.

Appendix G5 Westminster Borings Logs Compiled from As-Built Drawings and Geotechnical Reports

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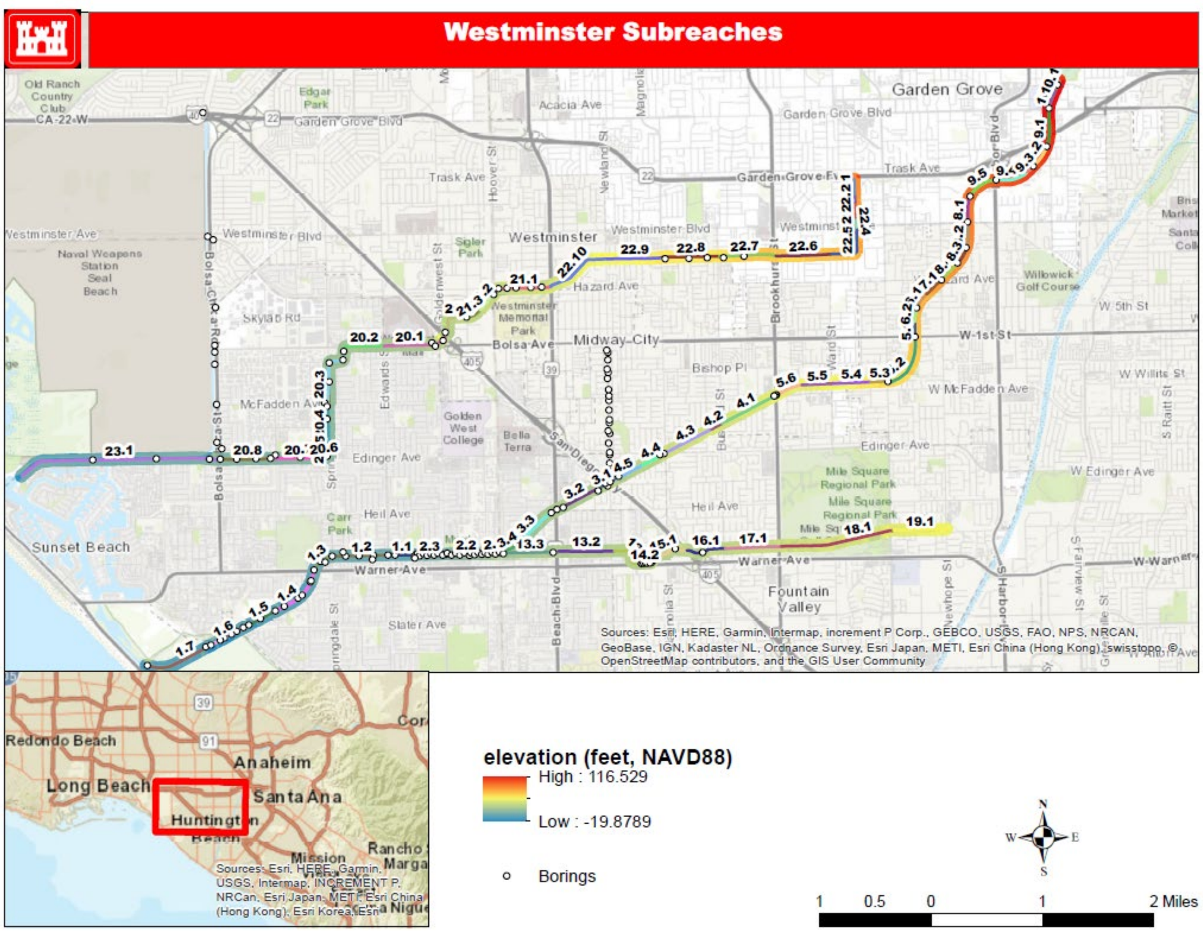


Figure 1. Borings by subreach (see attached for full scale)

Estimation of Soil Properties

The soil properties were estimated based on Unified Soil Classification System (USCS) descriptions and available blow counts. For clayey soils, the cohesion was estimated based on blow counts using a correlation developed for Chicago Soils as shown in Figure 2.

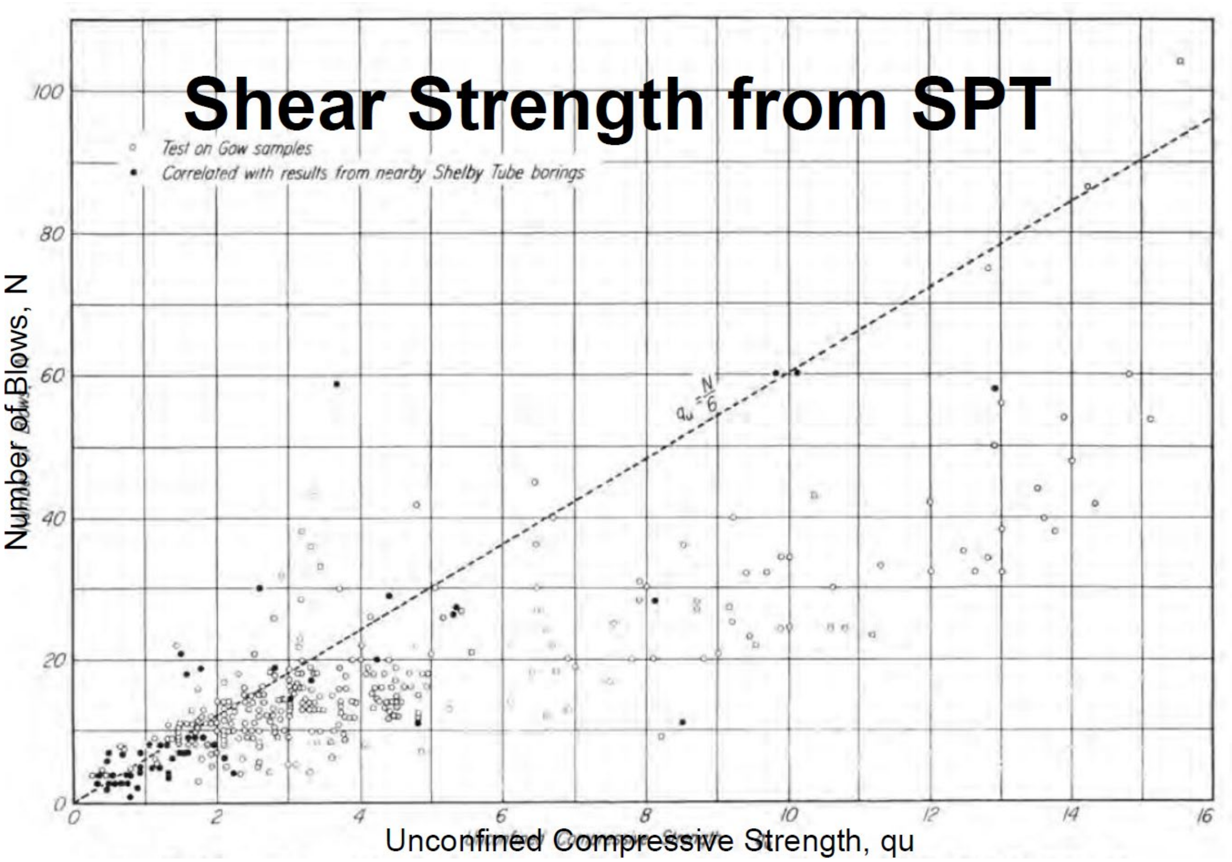


Figure 2. Shear strength from SPT blow counts (Peck and Reed, 1955)

The shear strength (c) can then be calculated as follows:

$$q_u = \frac{N}{6}\left(\frac{tons}{ft^2}\right)$$

$$c = \frac{q_u}{2} * \frac{2000\ lb}{1\ ton} = q_u * 1000 = \frac{N}{6} * 1000\ (\frac{lb}{ft^2})$$

Clay consistency can be estimated based on blow counts as shown in Figure 3. For low plasticity clays with soft to very soft consistency, the natural water content was estimated to be 40%, or roughly equivalent to the liquid limit of a low plasticity soils. For medium to stiff soils, the natural water content was estimated to be 30%. For stiff to hard low plasticity clays the natural water content was estimated to be 20%. Assuming a specific gravity of solids of 2.7, the unit weight of soils is estimated as shown in Table 1.

Sample Description

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT, additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits is estimated based on visual observation and is presented parenthetically on the test pit logs.

Density SAND or GRAVEL	Standard Penetration Resistance (N) in Blows/Foot	SILT or CLAY Consistency	Standard Penetration Resistance (N) in Blows/Foot	Approximate Shear Strength in TSF
Very loose	0 - 4	Very soft	0 - 2	<0.125
Loose	4 - 10	Soft	2 - 4	0.125 - 0.25
Medium dense	10 - 30	Medium stiff	4 - 8	0.25 - 0.5
Dense	30 - 50	Stiff	8 - 15	0.5 - 1.0
Very dense	>50	Very stiff	15 - 30	1.0 - 2.0
		Hard	>30	>2.0

Moisture

Dry	Little perceptible moisture
Damp	Some perceptible moisture, probably below optimum
Moist	Probably near optimum moisture content
Wet	Much perceptible moisture, probably above optimum

Minor Constituents	Estimated Percentage
Not identified in description	0 - 5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

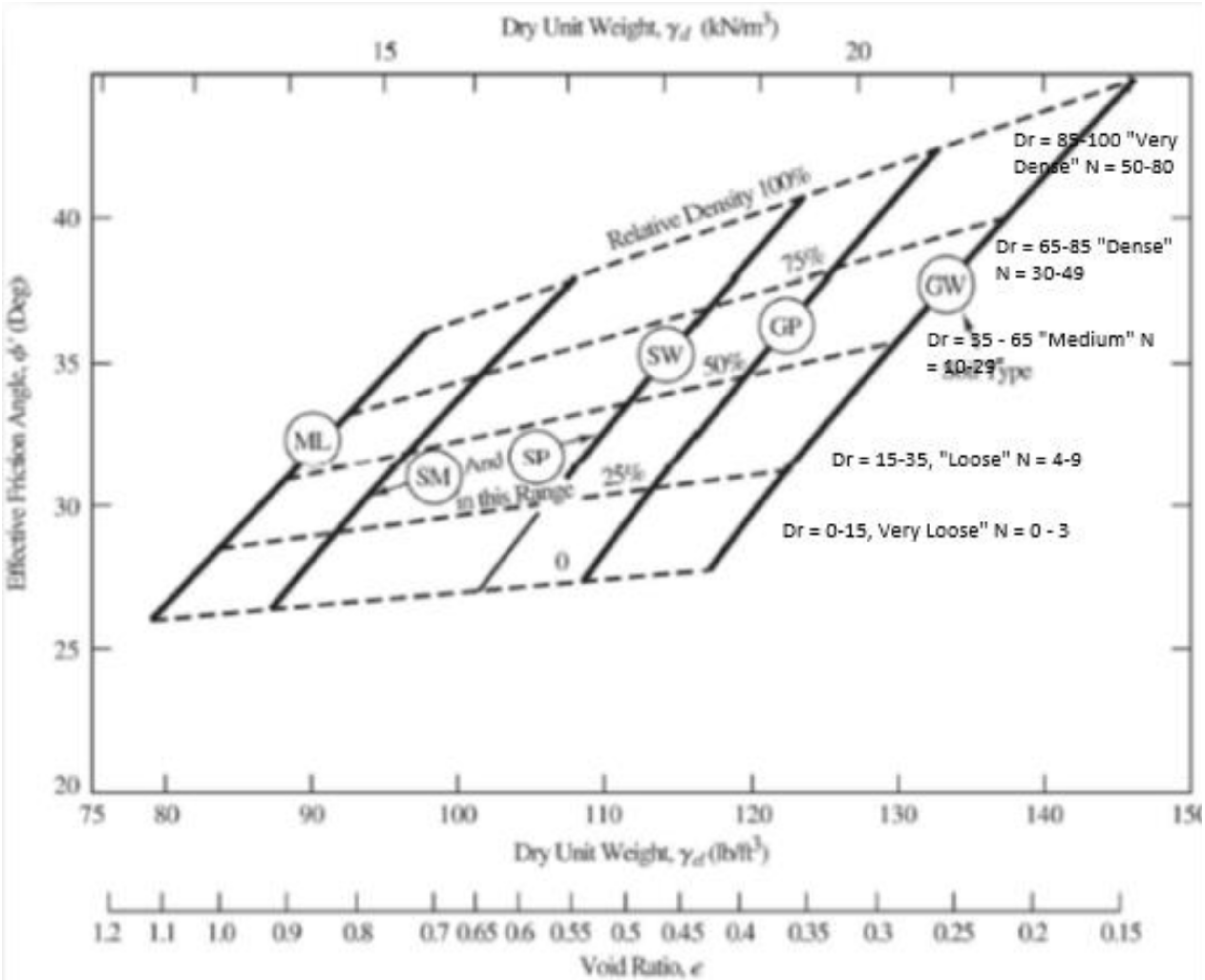
Figure 3. Soil consistency based on blow counts

Consistency	SPT blows per foot (N)	γ (pcf)
Very soft to soft	0 – 4	120
Medium stiff to stiff	4 - 15	125
Very stiff to hard	>15	130

Table 1. Estimation of unit weights of low plasticity soils based on blow counts

For granular soils (SP, SM, SW, GP, GM, GW, etc.) the dry unit weight and friction angle was estimated based on blow counts as shown in Figure 4. Granular soils were assumed to be saturated when estimating unit weight and moist unit weight was assumed to be the same as saturated unit weight. Saturated and moist unit weight is then calculated as follows

$$\gamma_{sat} = \gamma_m = \gamma_w - \frac{\gamma_d}{G_s} + \gamma_d$$



Description of Cross Sections

What follows is a description of the cross sections developed for the sub-reaches where data were available. Often blow counts were available in addition to USCS descriptions. Additionally borings were often available at regular intervals (less than 1000 ft) which permitted development of cross sections. To accurately develop these cross sections the blow counts were needed in addition to the USCS designations because often a hard/dense layer was discernable. This means that there were hard or dense layers that would affect geotechnical or structural design at the bottom of the borings that appear in multiple borings indicating that this is a consistent geologic layer.

Channel C02

Channel C02, which is the shortest channel and extends from the channel which ultimately exits in the Anaheim bay east to Bolsa Chica Road for this project. The levees in this area appear to be silt and silty sand, which can be erodible. The borings available in this area are missing blow counts, which would help characterize the strength and stratigraphic layers.

Subreach 23.1

Subreach 23.1 consists of a layer of silty clay and silt that ranges between 6 and zero feet, which overlies a 2 ft thick sand layer, overlaying a silty clay layer of zero to 3 feet at the channel centerline. Above the channel bottom there are silts, clays sands, silty clays, organic silts, and poorly graded sands. There were no available blow counts. As no structures are proposed in this area soil strength data may not be required for this subreach.

Channel C04

Channel C04 begins at Bolsa Chica Road and extends east and north as shown in Figure 1. Channel C04 consists of mainly inorganic soils. The main concern for this channel is lack of data. No geotechnical data were found for subreaches 22.10, 22.9 and 22.6 – 22.1. Additionally, the levees in this area appear to be silt and silty sand, which can be erodible.

Subreach 20.8 to 20.6

Subreach 20.8 to 20.6 has a thick layer of medium stiff to stiff low plasticity clay and silt to a depth of almost 40 ft, which thins to the east. At approximately 4000 ft from the west end of C04 the clay thins to approximately 10 ft and in underlain with a dense sand. There is a layer of high plasticity clay that appears in two borings in this area below the medium stiff to stiff clay.

Subreach 20.5 – 20.3

From subreach 20.5 through 20.3 there is a layer of medium stiff silt and low plasticity clay that is at least 20 ft thick.

Subreach 20.2 – 20.1

There are no available borings from subreach 20.2 – 20.1

Subreach 21.4 – 21.1

Subreach 21.4 to 21.1 consist mainly of medium stiff low plasticity clay, silt, and silty sand with a couple lenses of high plasticity clay.

Subreach 22.10 – 22.9

There are no available borings from subreach 22.10 – 22.9

Subreach 22.8 – 22.8

Add description

Subreach 22.6 – 22.1

There are no available borings from subreach 22.10 – 22.9

Channel C05

Channel C05 is the most geologically variable of the channels. It begins with a complex system of sand dunes, high plasticity clay, peat, and organic silt zones on the west end and becomes more homogenous and inorganic upstream beginning in subreach 4.5. Subreach 4.5 starts just east of the San Diego Freeway. Subreach 7.1 to the end at subreach 10.1 consists mainly of medium dense to dense poorly graded sands, and silty sands. Though there are numerous borings on the west end of Channel C05, there are a few subreaches without borings and many that are missing borings to adequately characterize the geology.

Subreach 1.7

Subreach 1.7 begins with clayey sand at the surface and transitions to thirty feet of sand on the west end of this subreach, which is approximately 3000 ft from the west end). The sand is dense, but the clayey sand is soft.

Subreach 1.6

Subreach 1.6 starts with hard silty clay and clayey sand and thins to the east. Under this layer is a 10 to 20 ft thick layer of medium dense sand, which is underlain by some pockets of hard clay. Within the hard clay layers is a layer of medium stiff silty clay which is then underlain by hard clay and dense sand

Subreach 1.5

Subreach 1.5 is similar to 1.6 with a thin layer of fine grained material (silty sand transitioning to silty clay) which is underlain by a medium dense sand, then a stiff silty clay. Under the silty clay is a layer of medium dense silty sand which transitions to dense sand to the east. There is a layer of well graded sand under the medium dense silty sand. A pocket of poorly graded gravel was found in B-02-12. At the bottom, roughly 50 ft below the bottom of the channel is a layer of dense sand.

Subreach 1.4

Subreach 1.4 has surface soils (5- 7 ft) of silt and poorly graded sands underlain by approximately 20 ft of silty clay to silty sand. A 15 ft thick layer of peat was found in boring B-04-1A which is located a little more than a third of the way from west to east. However peat was not found in other borings in this subreach. Under the fine grain materials is a layer of poorly graded sand. However blow counts for this sand were not available.

Subreach 1.3

Subreach 1.3 has 20 to 30 ft of organic silt and peat at the surface. There is a beach ridge approximately 2/3 the distance from the west of the subreach. Under the peat and organic silt is a layer of poorly graded sand. In the beach ridge area there is also interbedded high plasticity clay, peat, well graded gravel and sand. Below these layers is a dense sand from approximately -40 ft to -60 ft.

Subreach 1.2

Subreach 1.2 has a layer of loose silt and sand on the west end. There are two deposits of organic silt that extend 30 ft deep from the surface that are separated by what appears to be a beach ridge of medium dense poorly graded sand.

Subreach 1.1

There is a shallow deposit of peat to the west end of this reach. The rest of the reach is underlain by a medium dense sand to approximately – 40 ft. Then there is a layer of dense poorly graded then well graded sand. There are no borings in the middle to west end of this subreach.

Subreach 2.3

There is approximately 30 ft of organic silt on the west end of subreach 2.3 and a 20 ft thick layer of interbedded organic silt, sand, peat, high plasticity clay and silt on the east end. Between these two orgainic layers is an area of medium dense silt. Starting at a depth of approximately 20 ft the soils become harder and denser. About ¾ of the section on the west side is underlain by stiff to hard silts and clay from -20 to -40 ft. The remaining ¼ to the east is underlain by dense sands from -20 to -40 ft. Then the bottom of the subreach (-40 to -60) appears to be underlain by dense sands.

Subreach 2.2

The surface down approximately 10 ft for approximately the western 2/3 of the subreach consist of peats and interbedded peat, high plasticity clay silt and peat. The eastern 1/3 consists of medium dense to dense poorly graded sand. Below these layers from approximate elevation -10 to -30 there are stiff silts and clays. Below the silts and clays there are hard silts and clays interbedded with sand.

Subreach 2.1

Subreach 2.1 consists of stiff clays for the top 10 ft. There are no deep borings to the west end of the subreach. On the east end there are layers of high plasticity clay, silty clay, silt, low plasticity clay, silty sand, poorly graded sand and silt. The lower layer of sand is dense and the silt is stiff to hard.

Subreach 3.3

There is only one boring for subreach 3.3 and it appears to be offset from the centerline. There is organic silt for the top foot and then poorly graded sand for the next 2-3 ft. This subreach is missing borings for the west and center portion.

Subreach 3.2

Subreach 3.2 has a layer of organic silt on the west end that is nearly 10 ft deep. There is a ridge of silt and poorly graded sand to east of the organic silt. From a depth of 10 to nearly 20 ft there is a layer of stiff silty clay. This subreach is missing borings in the center and eastern portions.

Subreach 4.5

This subreach consists of poorly graded sand for the top 7 ft from the bottom of the channel. The boring was likely performed during the construction of I-405 and is offset from the channel alignment. There are no borings to the middle or eastern portion of this subreach.

Subreach 4.4

Subreach 4.4 consists of approximately 6 ft of silty clay underlain by 8 ft of poorly graded sand. The silty clay is stiff. This subreach is missing borings on the west and center portions.

Subreach 4.1

This subreach consists of medium dense silty sands, silts, some medium to stiff clay. From an elevation of 5 to -10 ft (bottom of boring) the sand and silty sand is dense.

Subreach 5.3

The top 12 ft or so consist of very stiff to stiff silty clay. The bottom 6 ft consist of hard silty clay. There are no borings

Subreach 5.2

No borings for this subreach.

Subreach 5.1

To the east end of this subreach there is a boring showing 5 – 6 feet of loose sand which overlays 12 feet of stiff to very stiff silty clay. There are no borings to west and center portions of this subreach.

Subreach 6.2

No borings for this subreach.

Subreach 6.1

There is one boring toward the western side of this subreach that consists of poorly graded sand for the top 2 – 3 ft, then 2 feet of stiff silty clay, which overlies 12 feet of dense poorly graded sand.

Subreach 7.1 to 10.1

This subreach consists mainly of poorly graded sand. Borings in this area extend approximately 10 ft below the bottom of the channel. The sand ranges from medium dense to dense.

C06

Channel C06 begins at the confluence of Channel C05 and C06 just east of Gothard and a rail line. Channel C06 has only a few borings with many of the borings lacking blow counts that would help characterize the soil strength and stratigraphy.

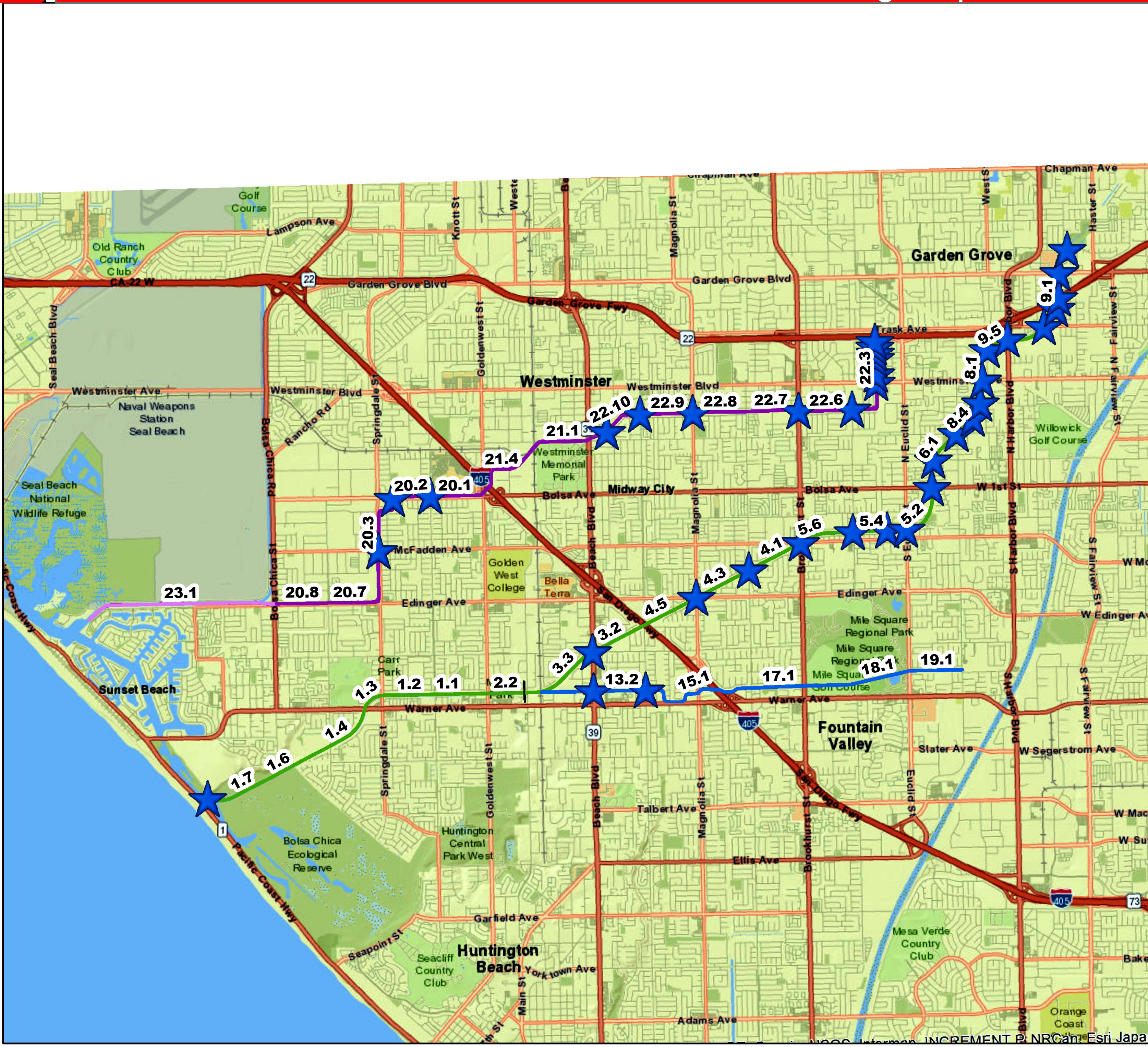
Subreach 13.3 – 13.1

There are two borings to west end of this subreach. The western most borings stiff to very stiff silty clay for the top 10 feet underlain by medium dense well graded sand. The boring approximately 1/3 from the western end has an approximately 7 ft thick layer of organic silt underlain by silty clay. This subreach is missing borings to the center and eastern portion.

Subreach 14.2 to 16.1

There were a number of borings to the western portion of these subreaches that show silt and silty clay in the leveed area and then clay and silty sand in the area beneath the channel bottom. There is one boring in the central portion of this subreach that shows silt. The boring in the center shows the silt to be very loose. However there are no blow counts for the borings to the west of this subreach.

Westminster Channel Subreaches and Crossing Replacements for LPP



Crossing Replacment

Channel

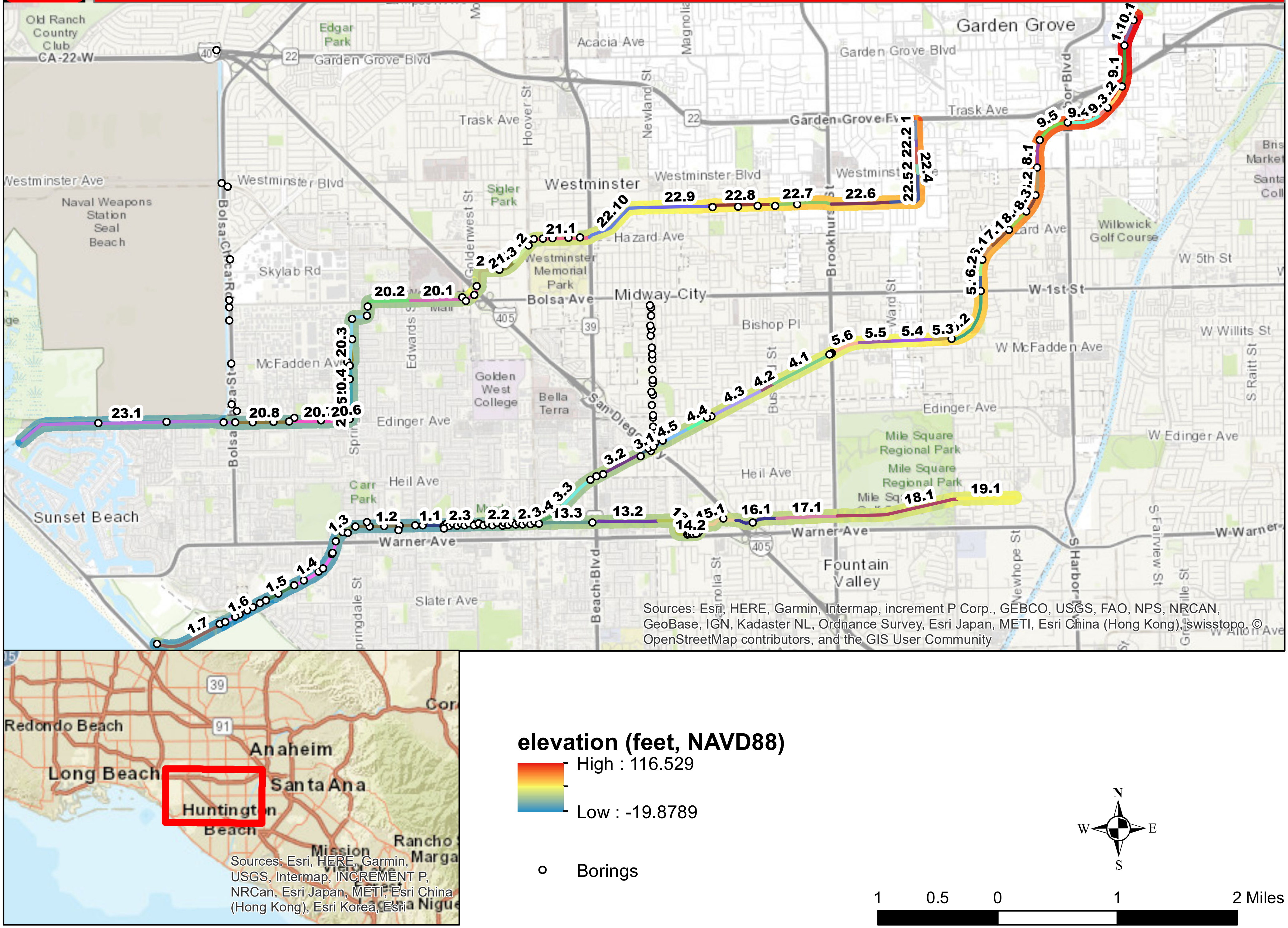
- C02
- C04
- C05
- C06

Location Map

1.25 0.625 0 Miles

1 in = 1.25 miles

Westminster Subreaches



Chicago District
231 S Lasalle
Chicago, IL 60604

KEY TO SYMBOLS

CLIENT ______

PROJECT NUMBER ______

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

- CH: USCS High Plasticity Clay
- CL: USCS Low Plasticity Clay
- CL-ML: USCS Low Plasticity Silty Clay
- FILL: Fill (made ground)
- GW: USCS Well-graded Gravel
- MH: USCS Elastic Silt
- ML: USCS Silt
- OH: USCS High Plasticity Organic silt or clay
- OL: USCS Low Plasticity Organic silt or clay
- PT: USCS Peat
- SC: USCS Clayey Sand
- SC-SM: USCS Clayey Sand
- SM: USCS Silty Sand
- SP: USCS Poorly-graded Sand
- SP-SM: USCS Poorly-graded Sand with Silt
- SW: USCS Well-graded Sand

SAMPLER SYMBOLS

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL - LIQUID LIMIT (%)
PI - PLASTIC INDEX (%)
W - MOISTURE CONTENT (%)
DD - DRY DENSITY (PCF)
NP - NON PLASTIC
-200 - PERCENT PASSING NO. 200 SIEVE
PP - POCKET PENETROMETER (TSF)

TV - TORVANE
PID - PHOTOIONIZATION DETECTOR
UC - UNCONFINED COMPRESSION
ppm - PARTS PER MILLION

Water Level at Time Drilling, or as Shown
Water Level at End of Drilling, or as Shown
Water Level After 24 Hours, or as Shown

Chicago District
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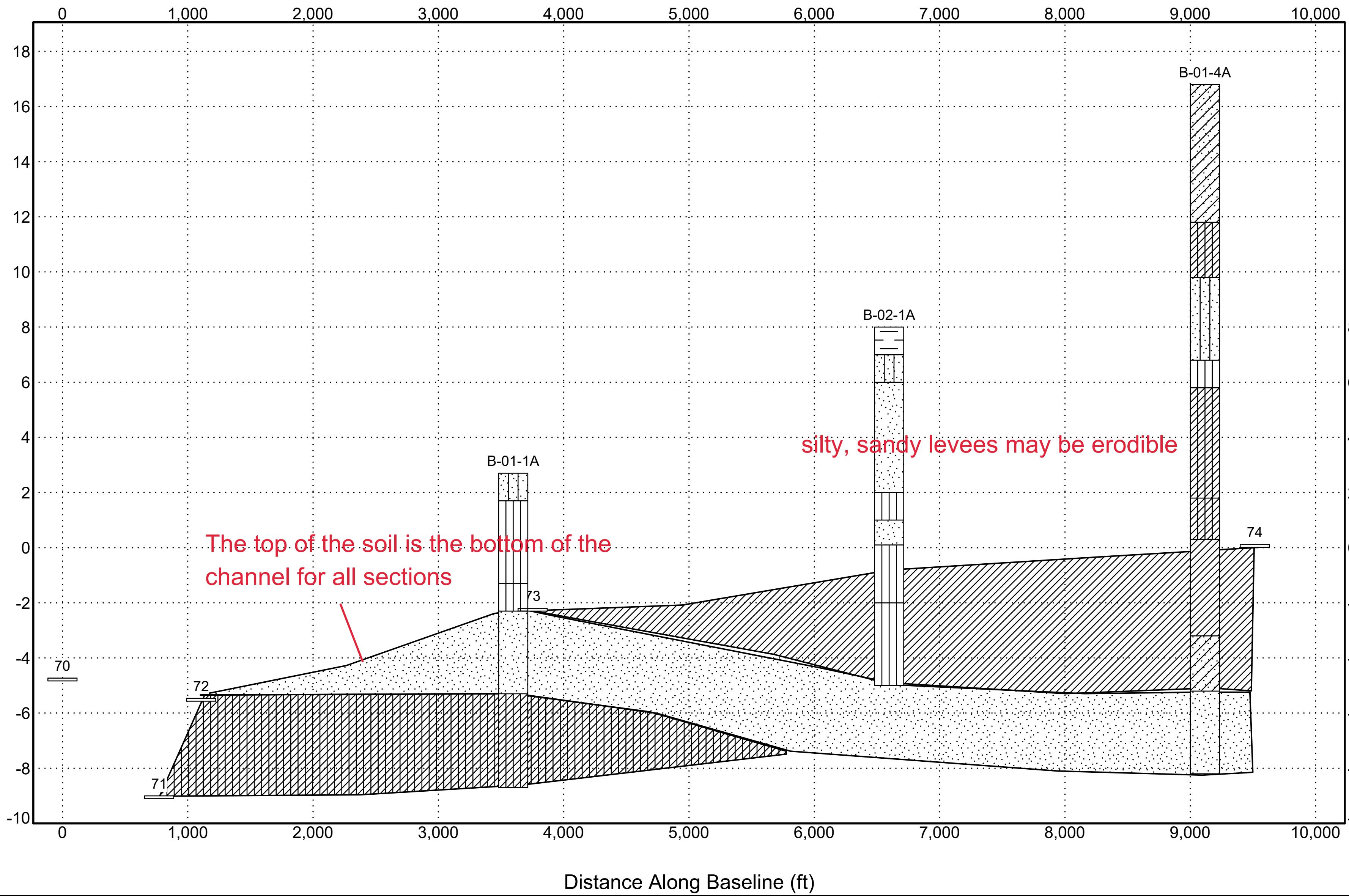
SUBSURFACE DIAGRAM
Subreach 23.1 (offset 125 ft)

CLIENT ______

PROJECT NUMBER ______

PROJECT NAME Westminster Bolsa Chica, Channel C02

PROJECT LOCATION ______



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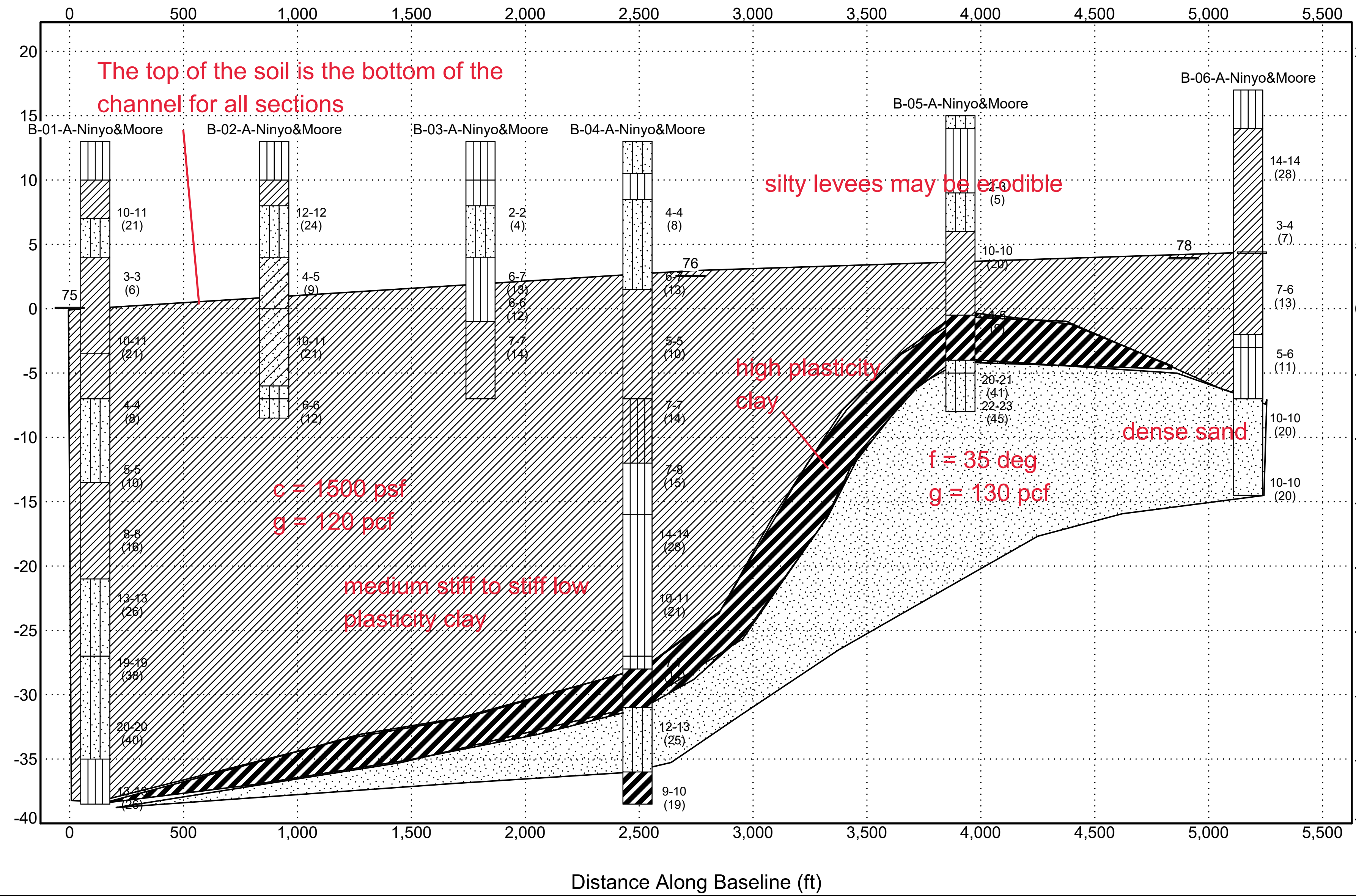
SUBSURFACE DIAGRAM
Subreach 20.8 to 20.6 (125 ft offset)

CLIENT ______

PROJECT NUMBER ______

PROJECT NAME Westminster, Channel C04

PROJECT LOCATION ______



SUBSURFACE DIAGRAM
Subreach 20.5 to 20.3 (offset 125 ft)

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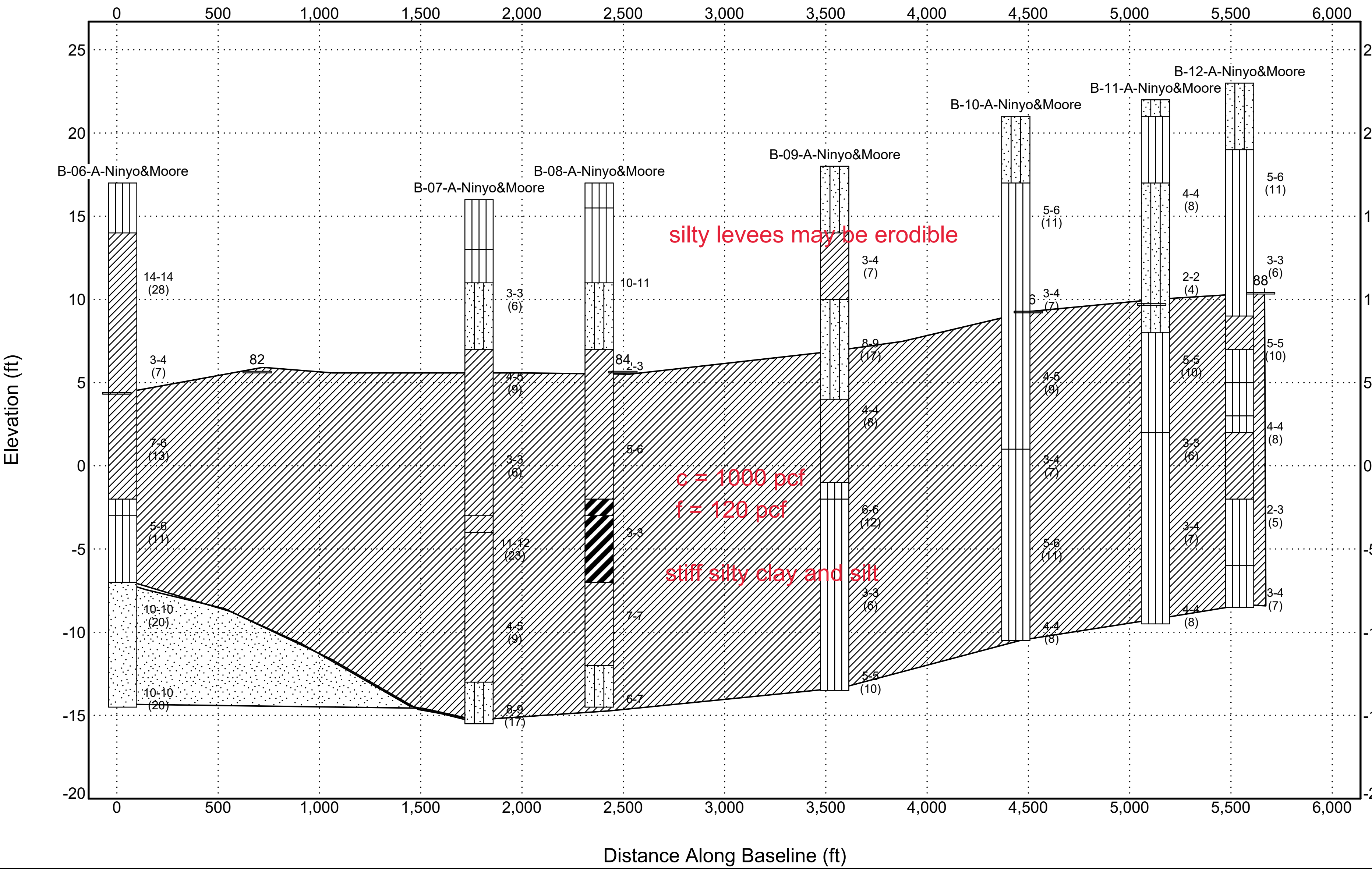
CLIENT

PROJECT NUMBER

PROJECT NAME Westminster, Channel C04

PROJECT LOCATION



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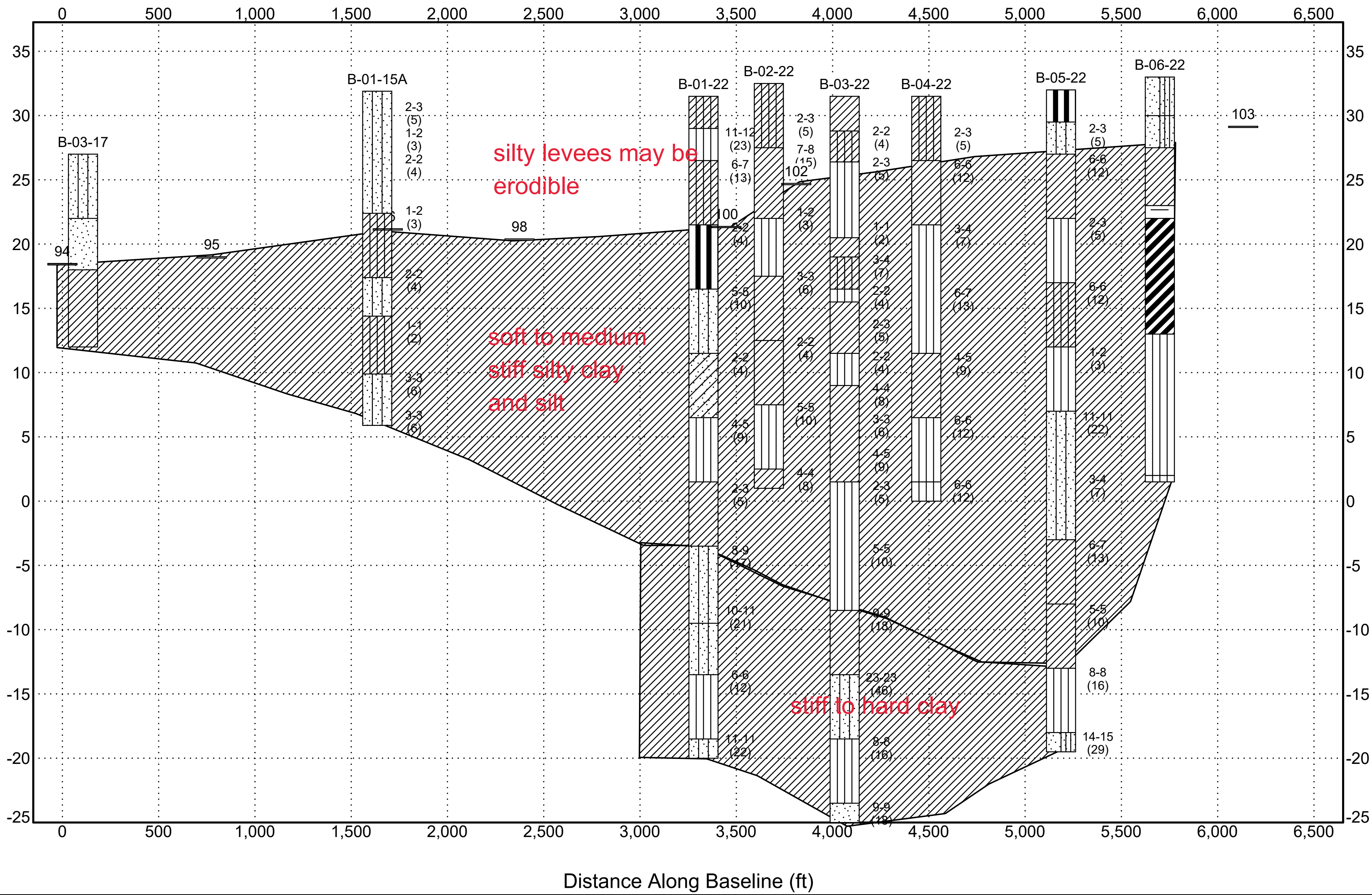
SUBSURFACE DIAGRAM
Subreach 21.4 to 21.1 (offset 125 ft)

CLIENT ______

PROJECT NUMBER ______

PROJECT NAME Westminster, Channel C04

PROJECT LOCATION ______



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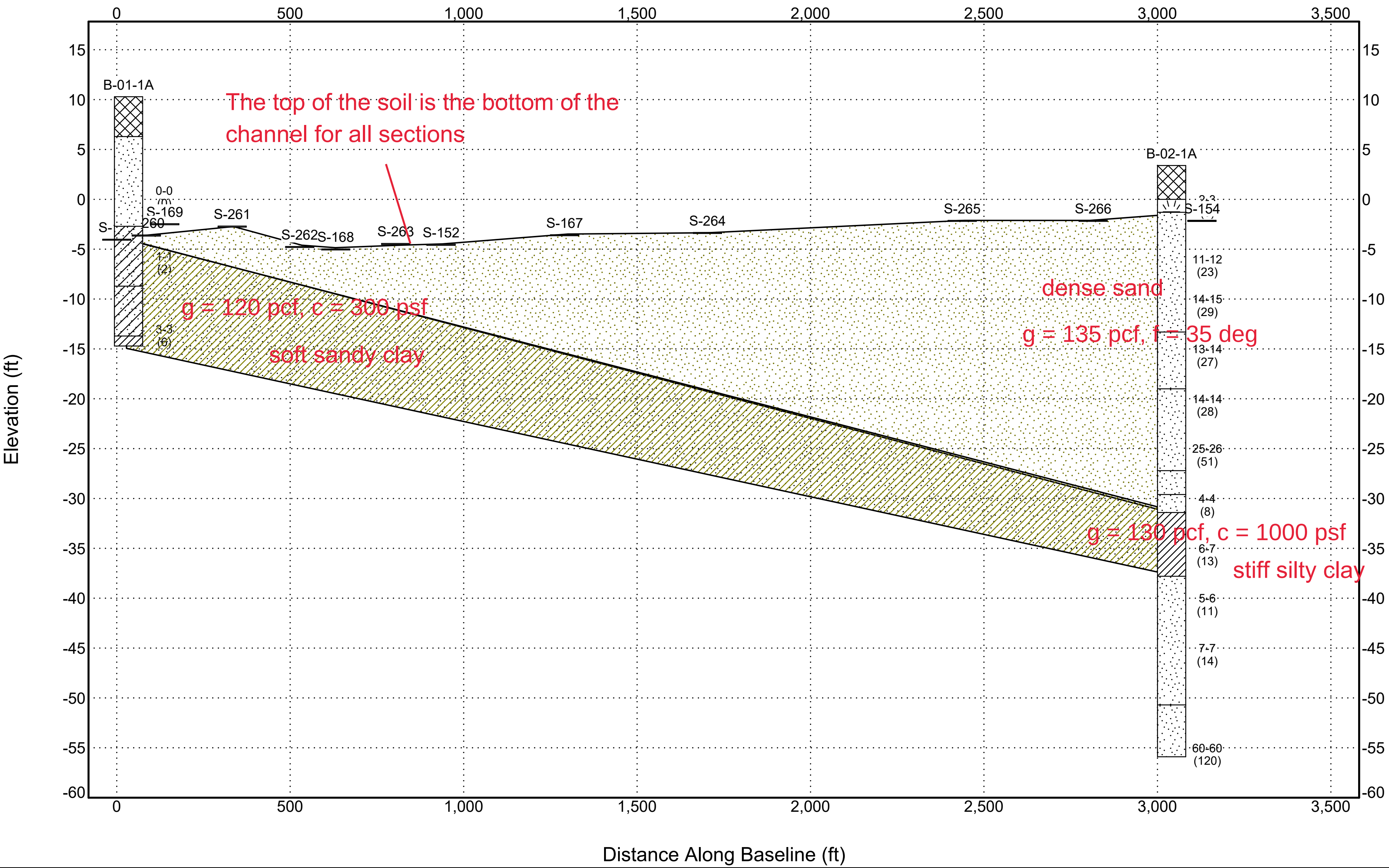
SUBSURFACE DIAGRAM
Subreach 1.7

CLIENT ______________________

PROJECT NUMBER ______________________

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______________________



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SUBSURFACE DIAGRAM
Subreach 1.6

US Army Corps of Engineers

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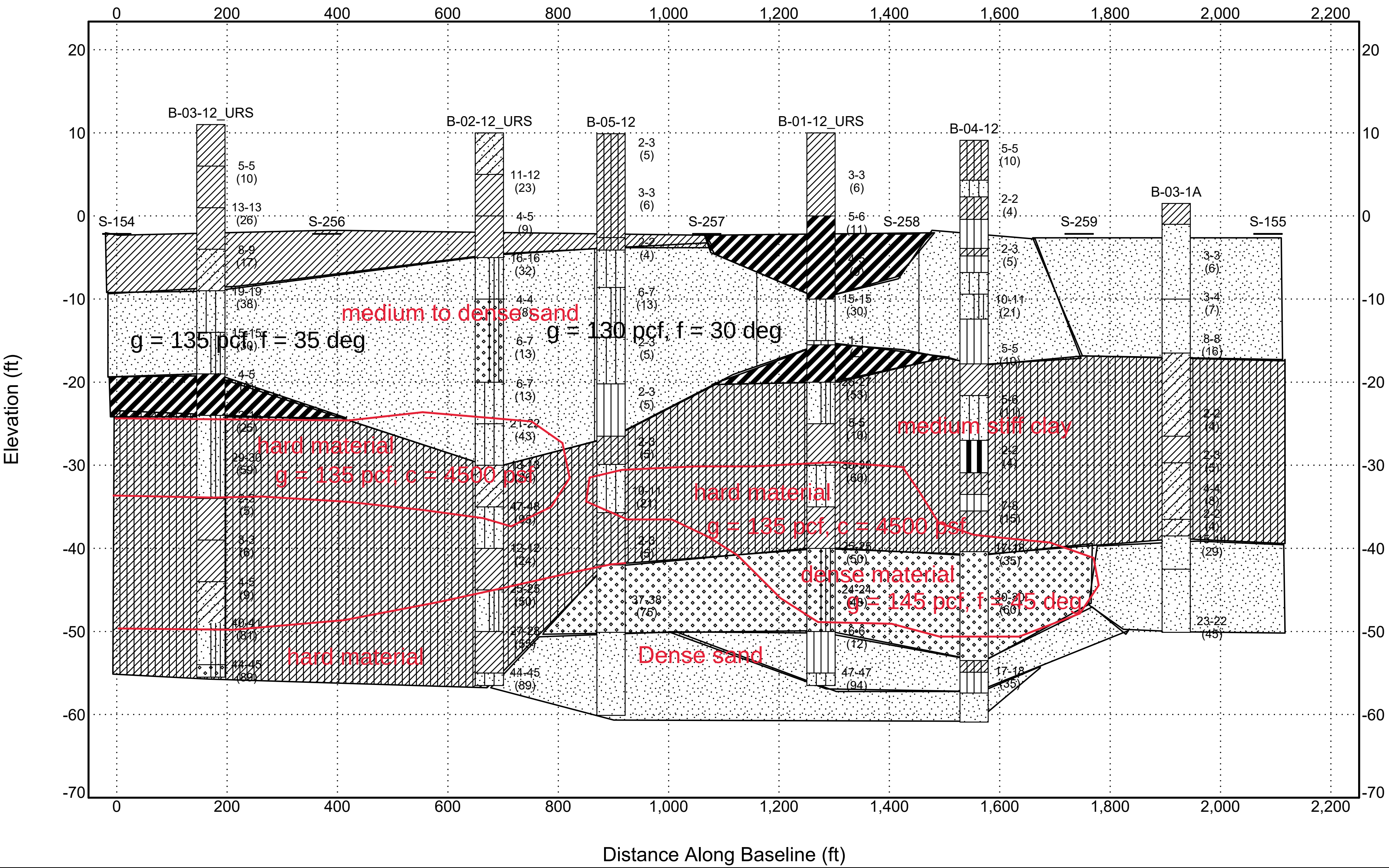
Chicago, IL 60604

CLIENT ______________________

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT NUMBER ______________________

PROJECT LOCATION ______________________



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Chicago, IL 60604

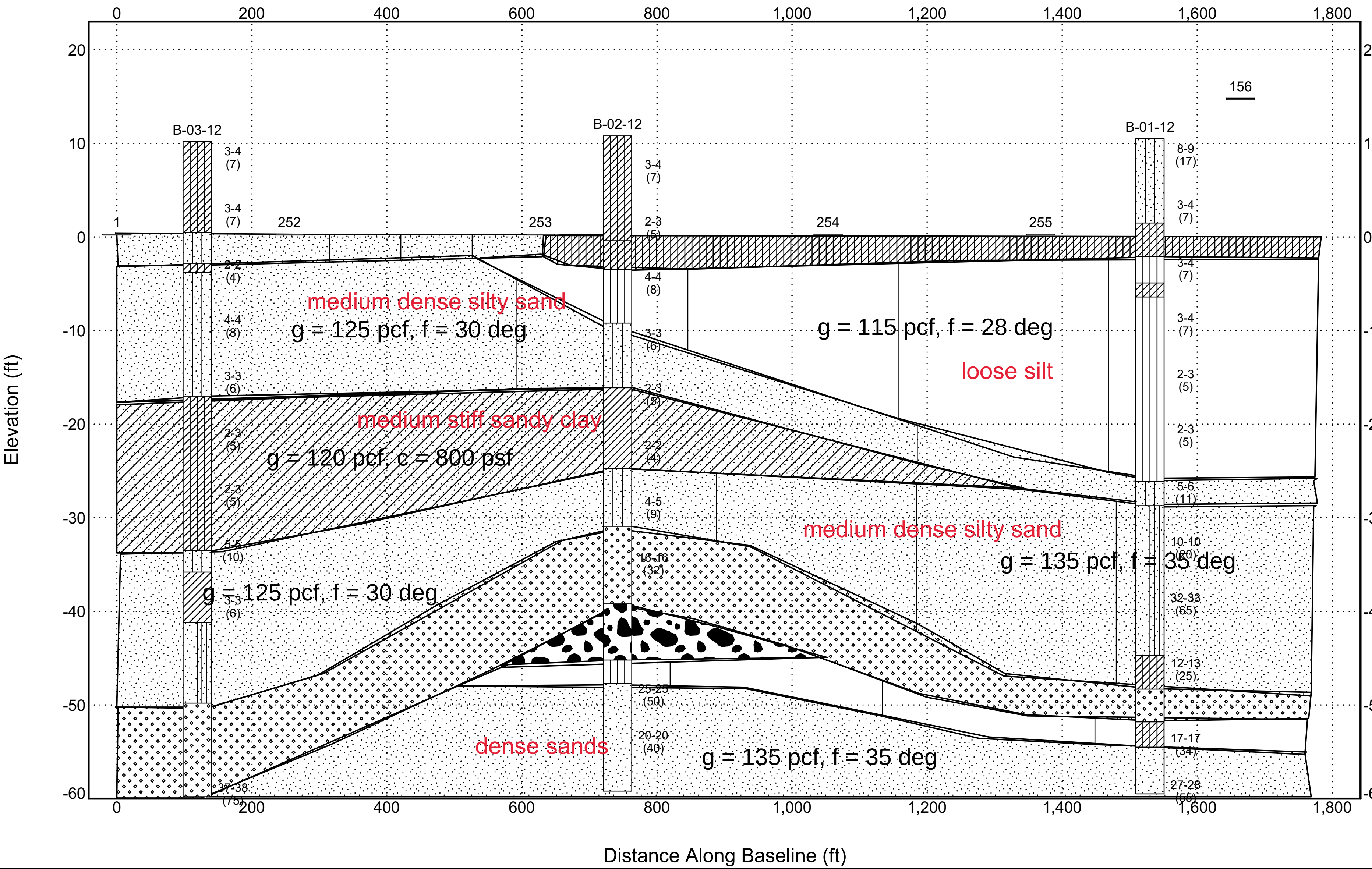
SUBSURFACE DIAGRAM
Subreach 1.5

CLIENT ______

PROJECT NUMBER ______

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______



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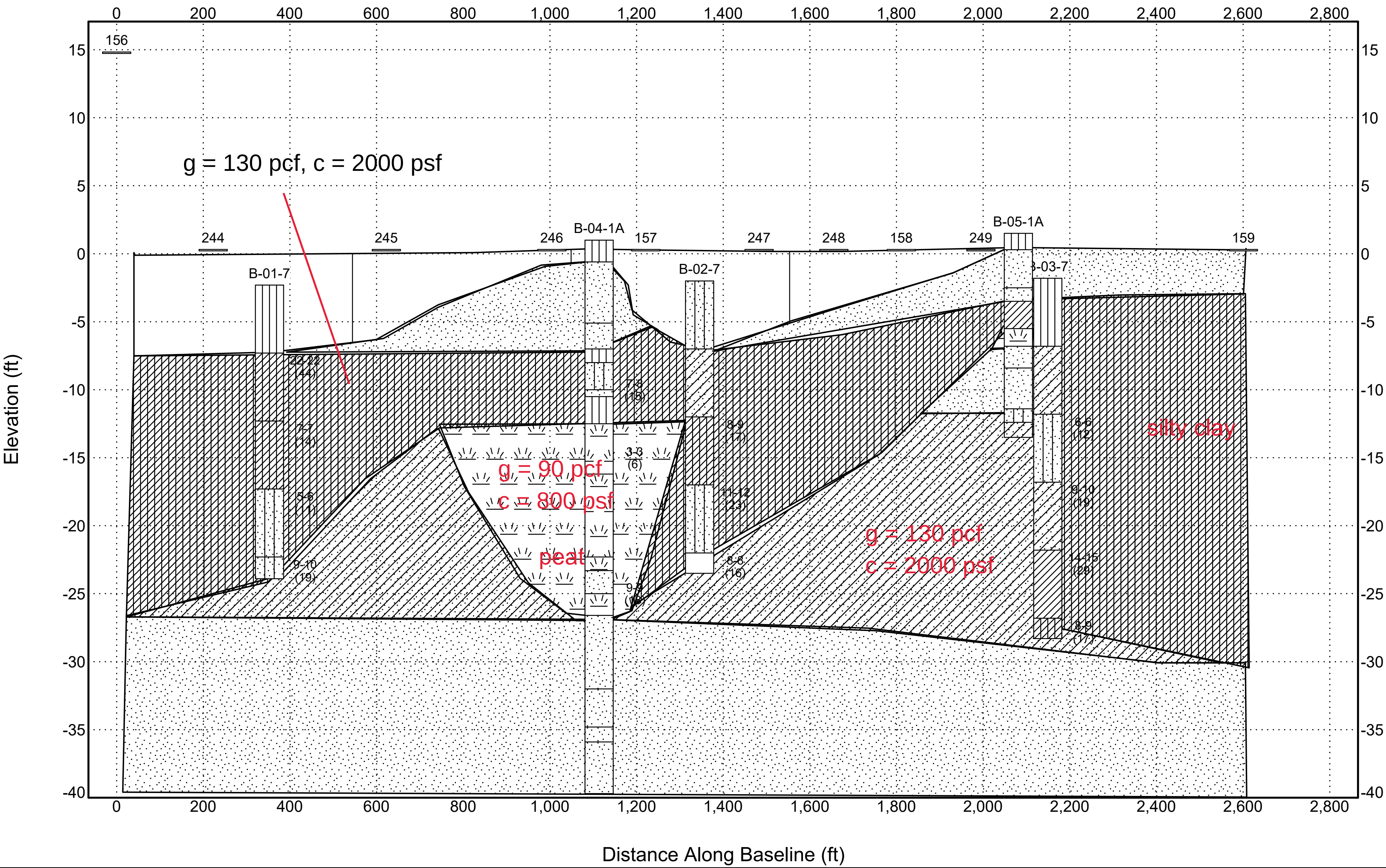
SUBSURFACE DIAGRAM
Subreach 1.4

CLIENT ______________________

PROJECT NUMBER ______________________

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______________________



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SUBSURFACE DIAGRAM
Subreach 1.3

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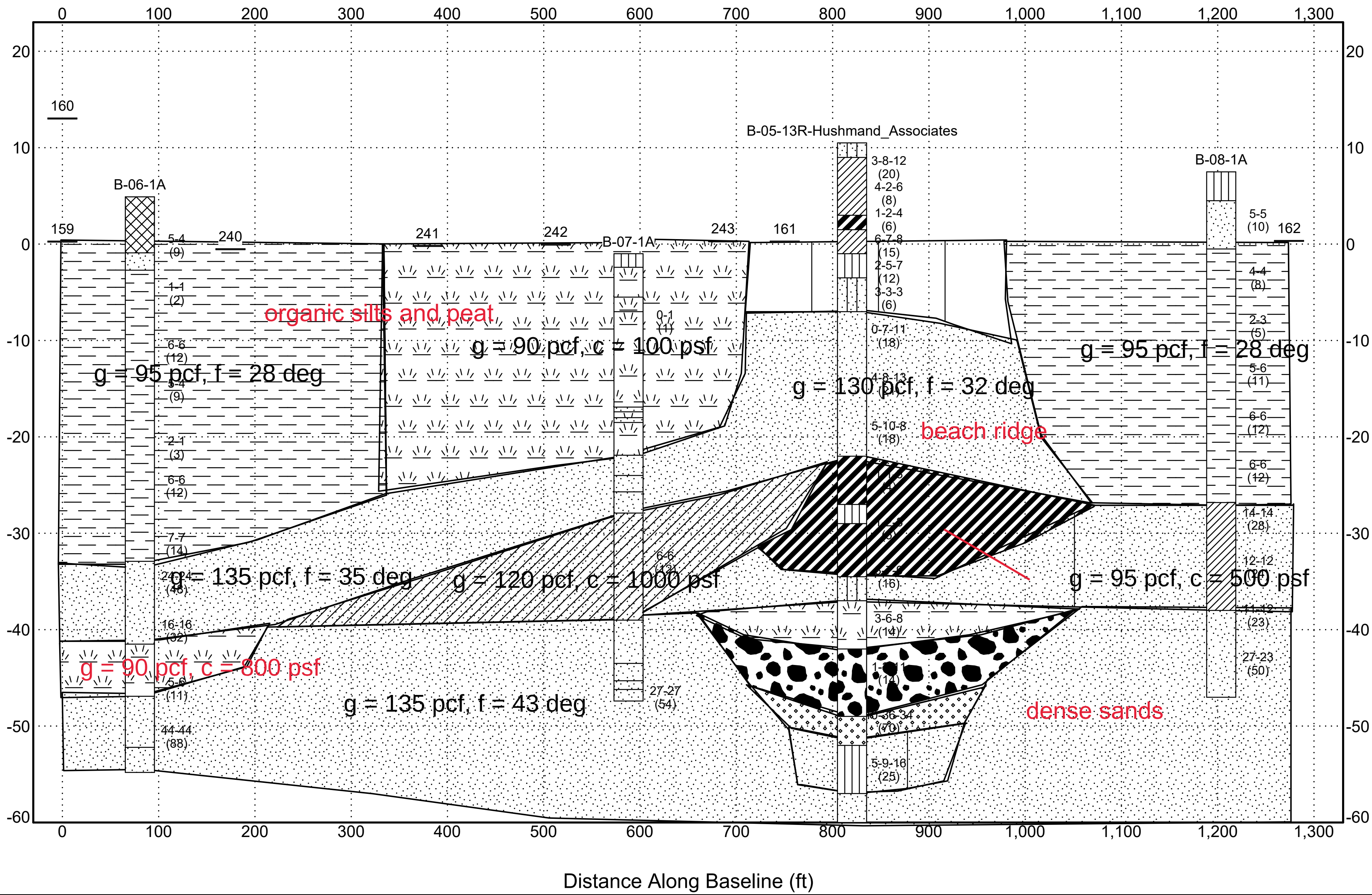
CLIENT ______

PROJECT NUMBER ______

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______



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Chicago, IL 60604

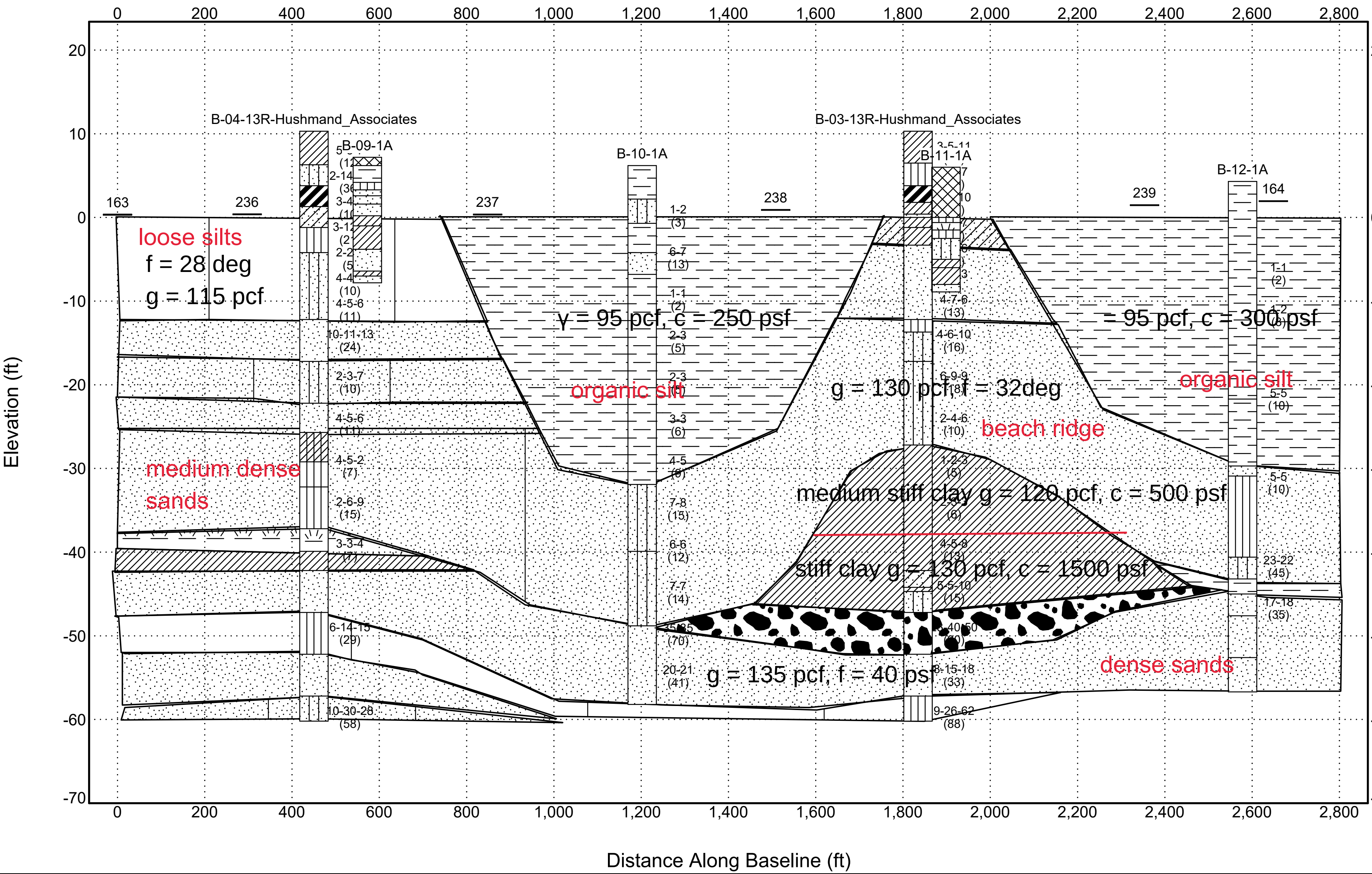
SUBSURFACE DIAGRAM
Subreach 1.2

CLIENT ______

PROJECT NUMBER ______

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______



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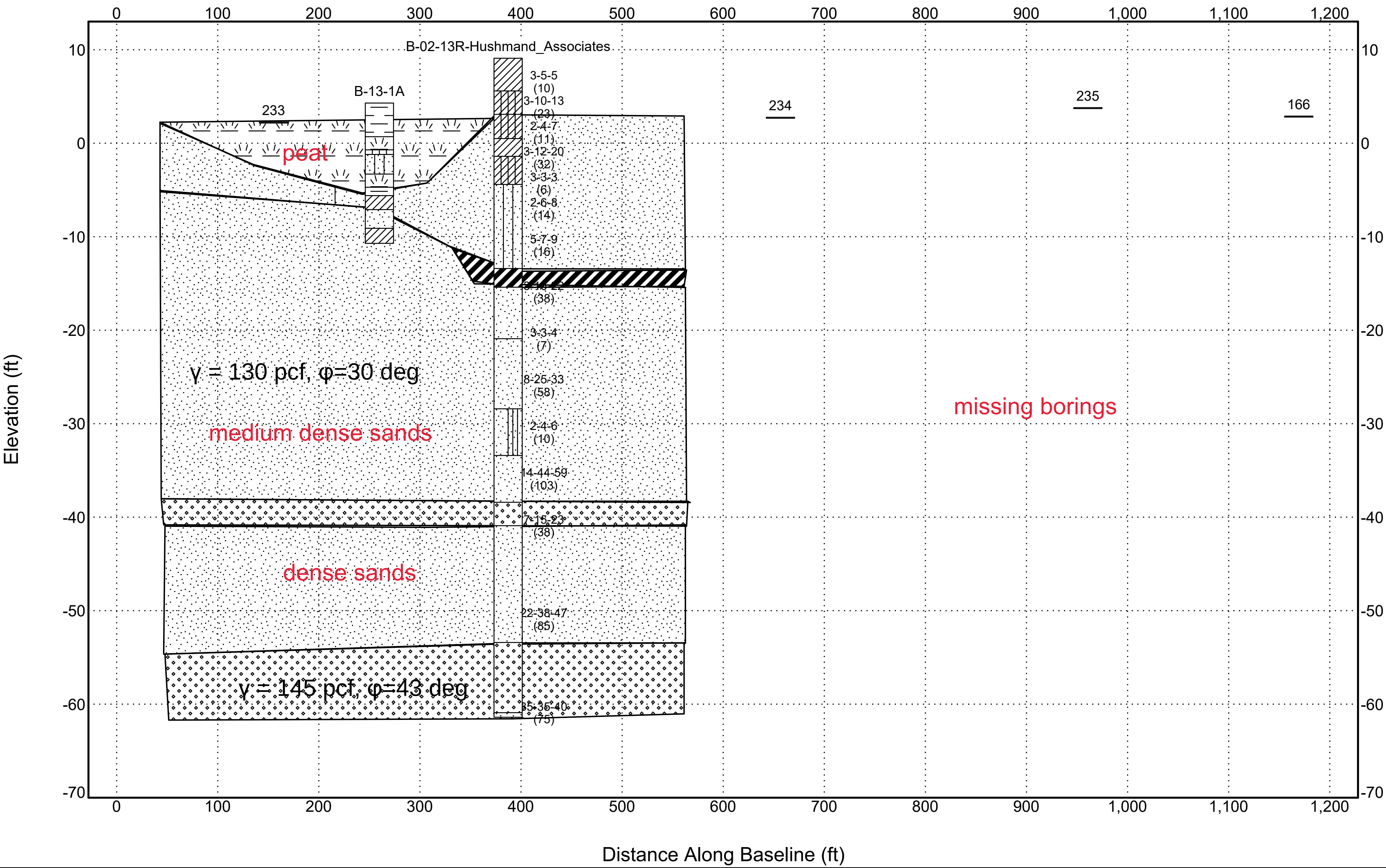
SUBSURFACE DIAGRAM
Subreach 1.1

CLIENT

PROJECT NUMBER

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION



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SUBSURFACE DIAGRAM
Subreach 2.3

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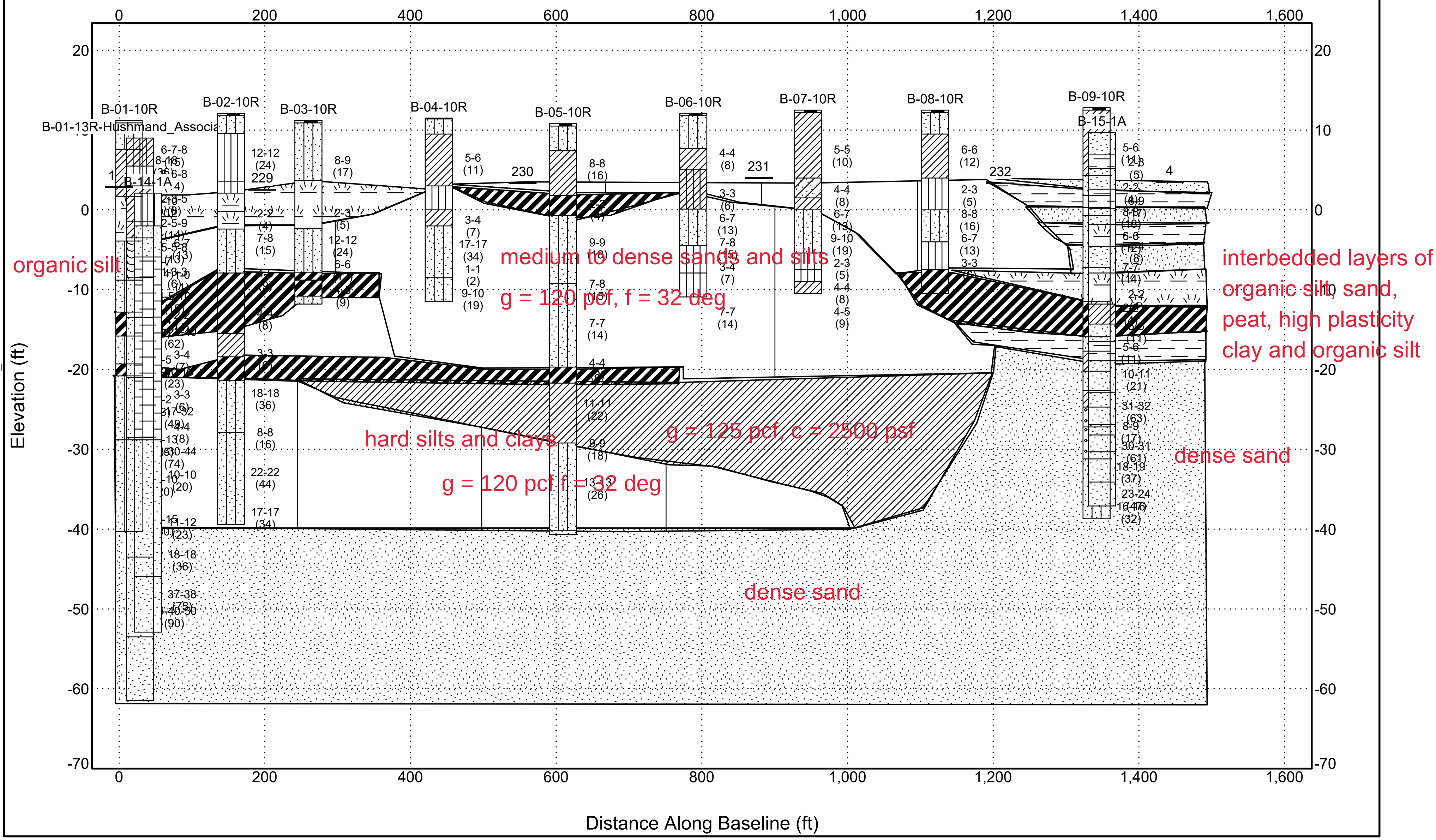
CLIENT ______________________

PROJECT NUMBER ______________________

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______________________



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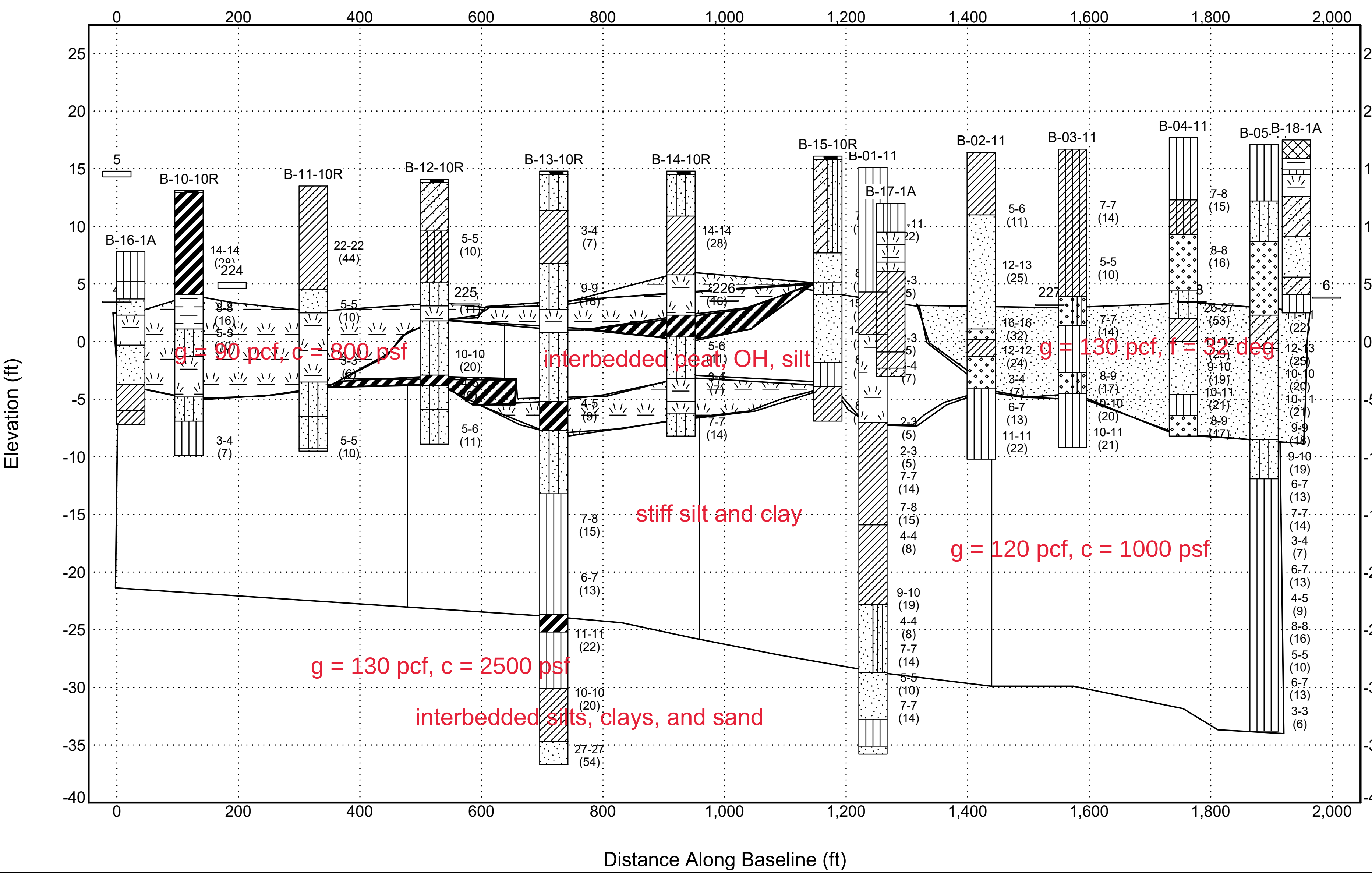
SUBSURFACE DIAGRAM
Subreach 2.2

CLIENT ______________________

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT NUMBER ______________________

PROJECT LOCATION ______________________



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Chicago, IL 60604

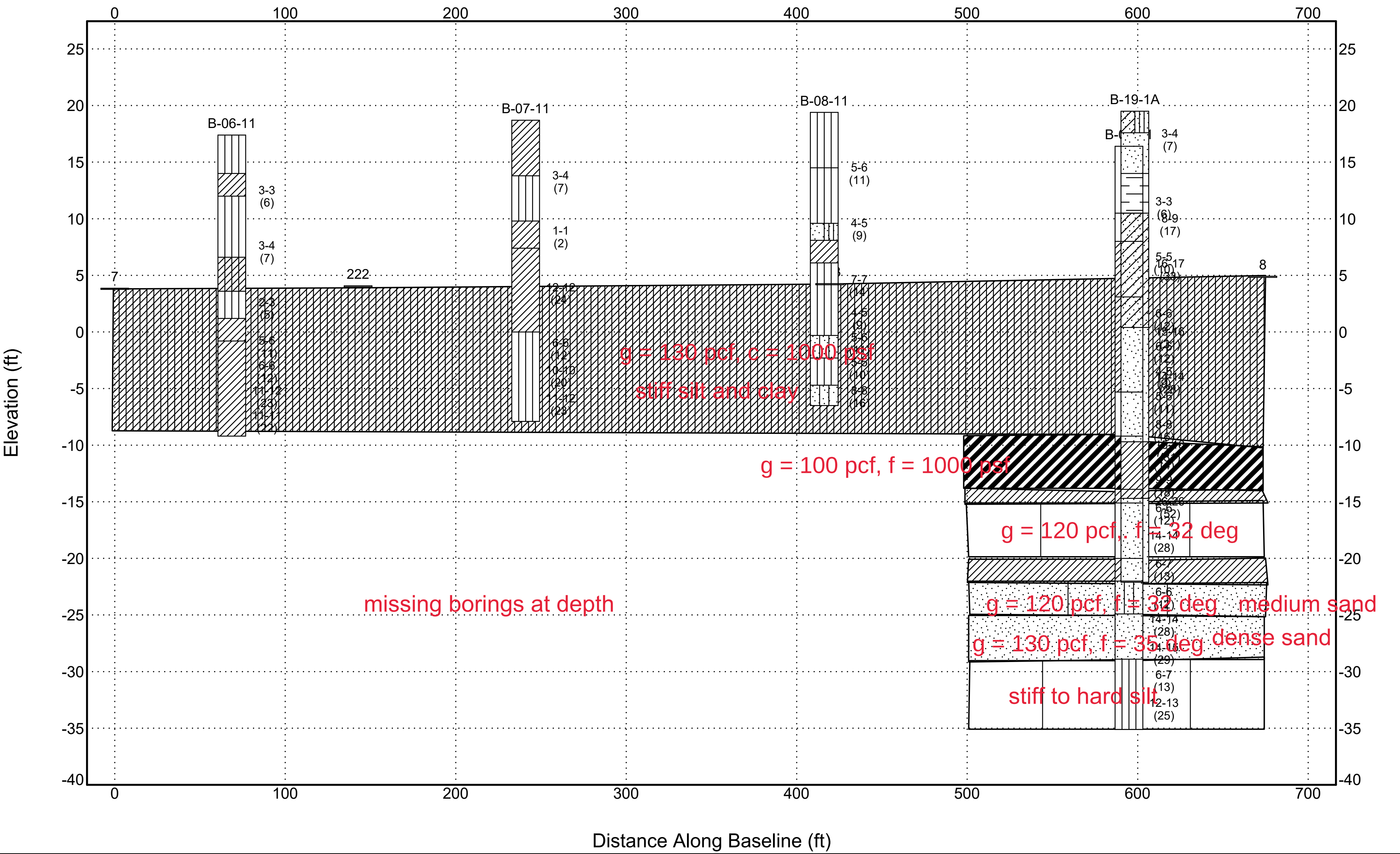
SUBSURFACE DIAGRAM
Subreach 2.1

CLIENT ______________________

PROJECT NUMBER ______________________

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______________________



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Chicago, IL 60604

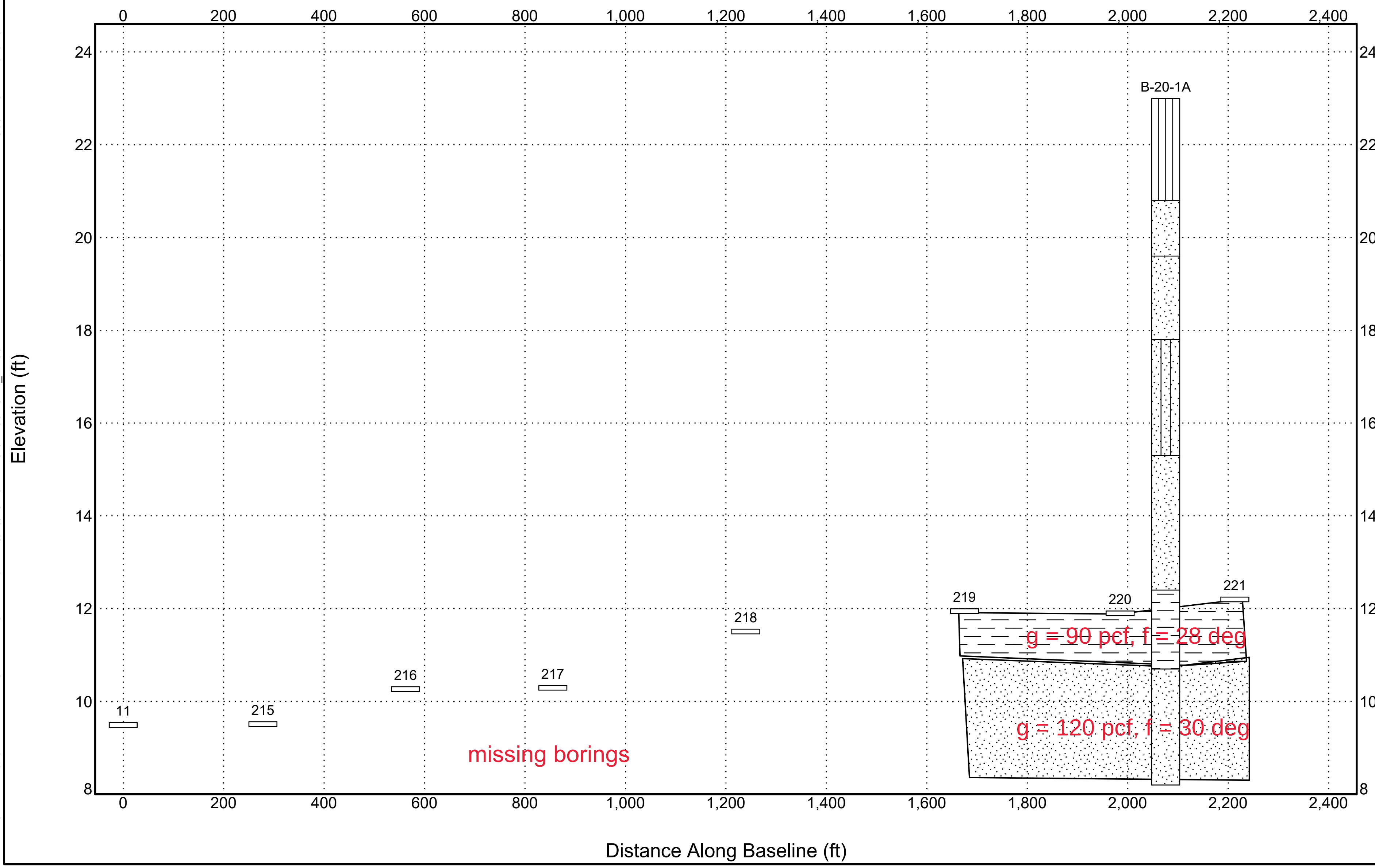
SUBSURFACE DIAGRAM
Subreach 3.3

CLIENT ______

PROJECT NUMBER ______

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______



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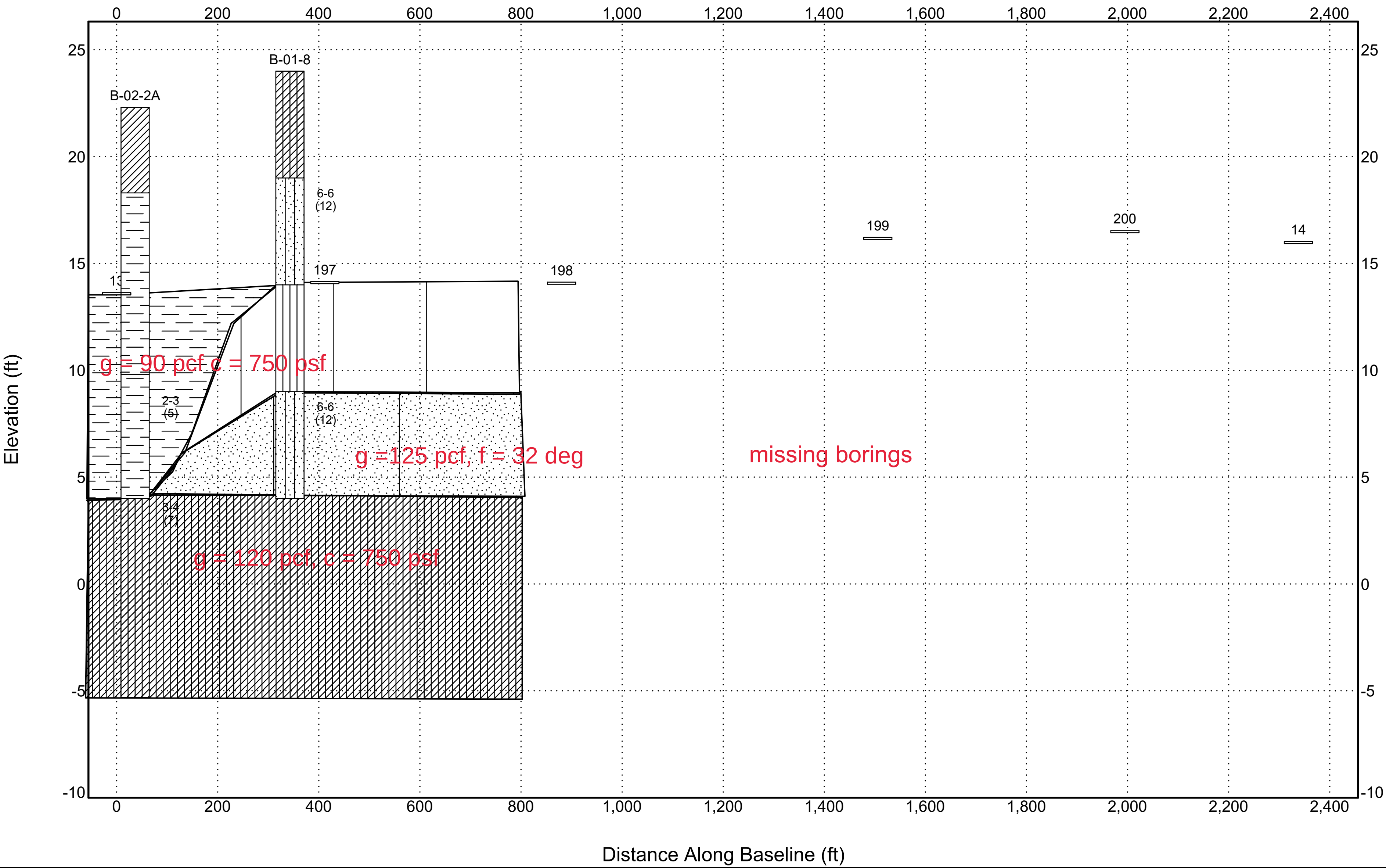
SUBSURFACE DIAGRAM
Subreach 3.2

CLIENT ______________________

PROJECT NUMBER ______________________

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______________________



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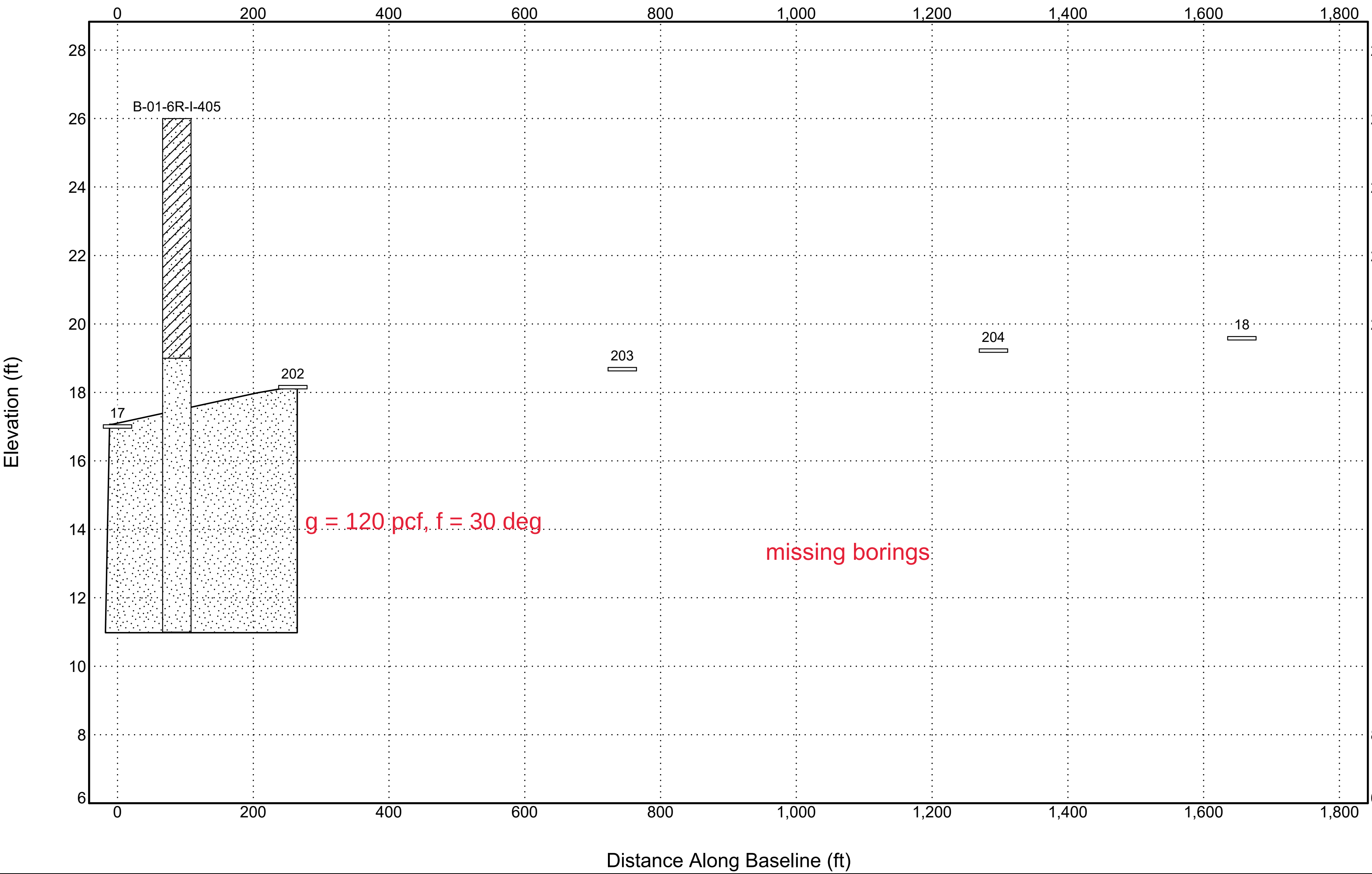
SUBSURFACE DIAGRAM
Subreach 4.5

CLIENT ______

PROJECT NUMBER ______

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______



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Chicago District
231 S Lasalle
Chicago, IL 60604

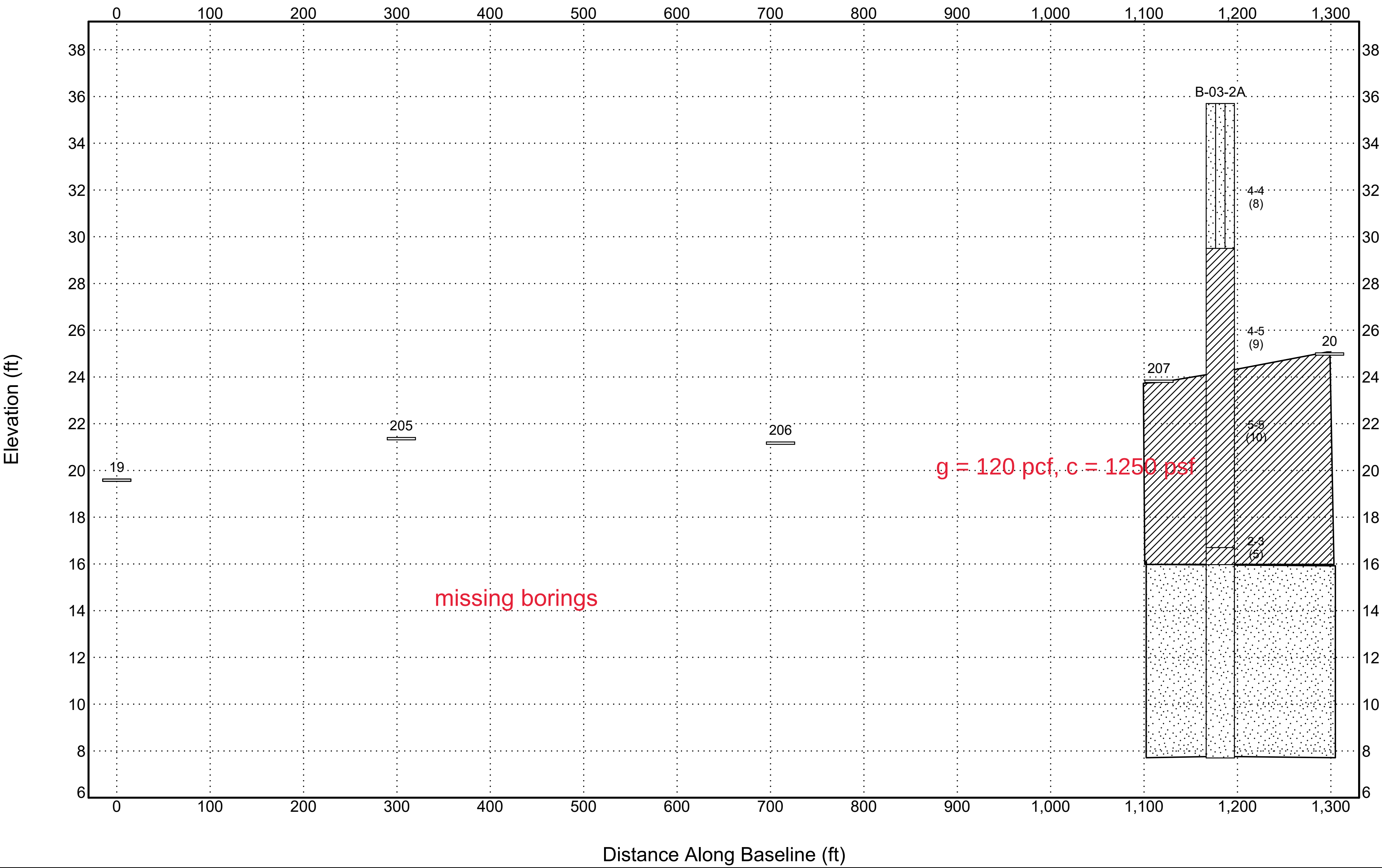
SUBSURFACE DIAGRAM
Subreach 4.4 (125 ft offset)

CLIENT ______________________

PROJECT NUMBER ______________________

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______________________



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Chicago District
231 S Lasalle
Chicago, IL 60604

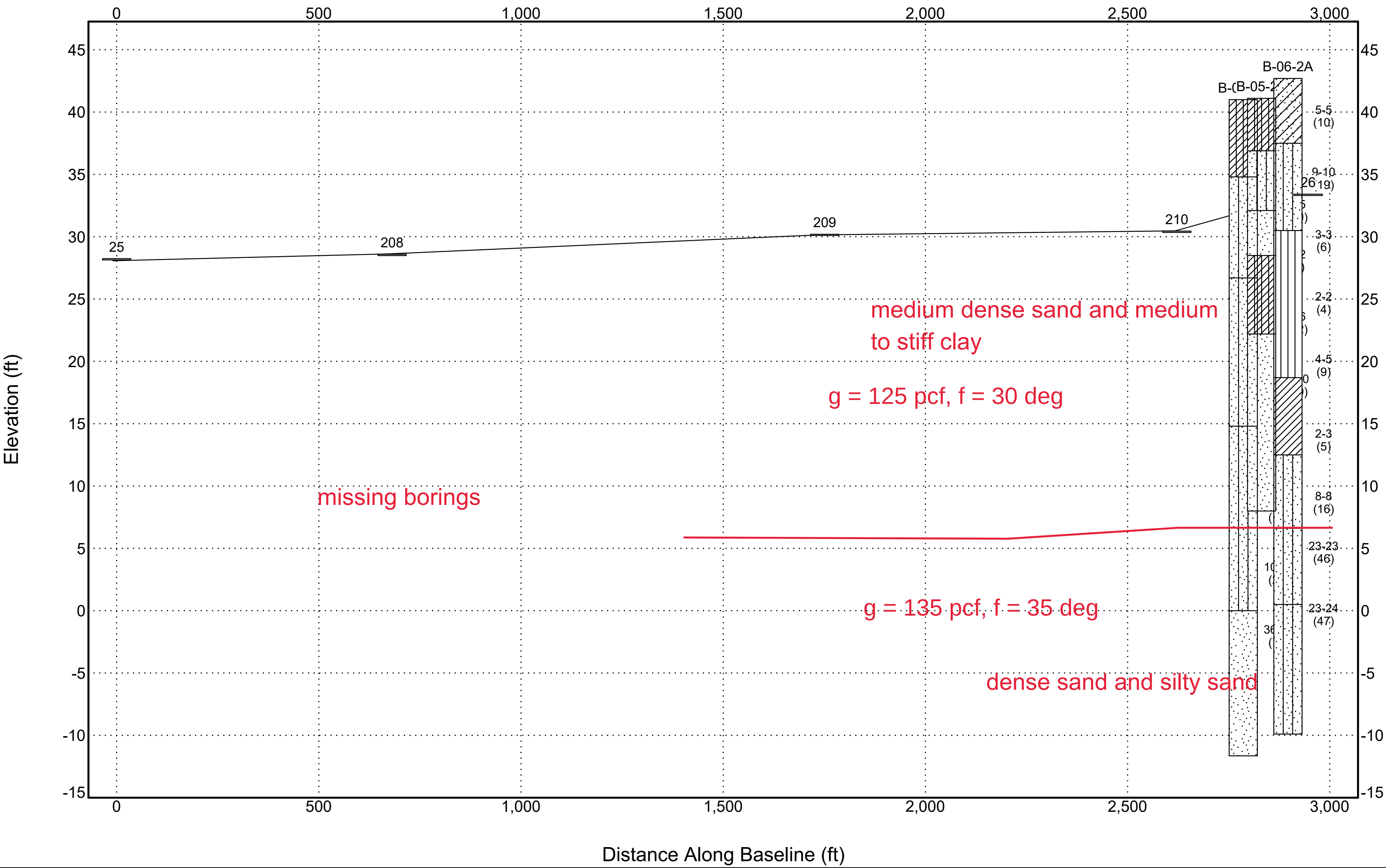
SUBSURFACE DIAGRAM
Subreach 4.1 (125 ft offset)

CLIENT ______________________

PROJECT NUMBER ______________________

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______________________



Chicago District
231 S Lasalle
Chicago, IL 60604

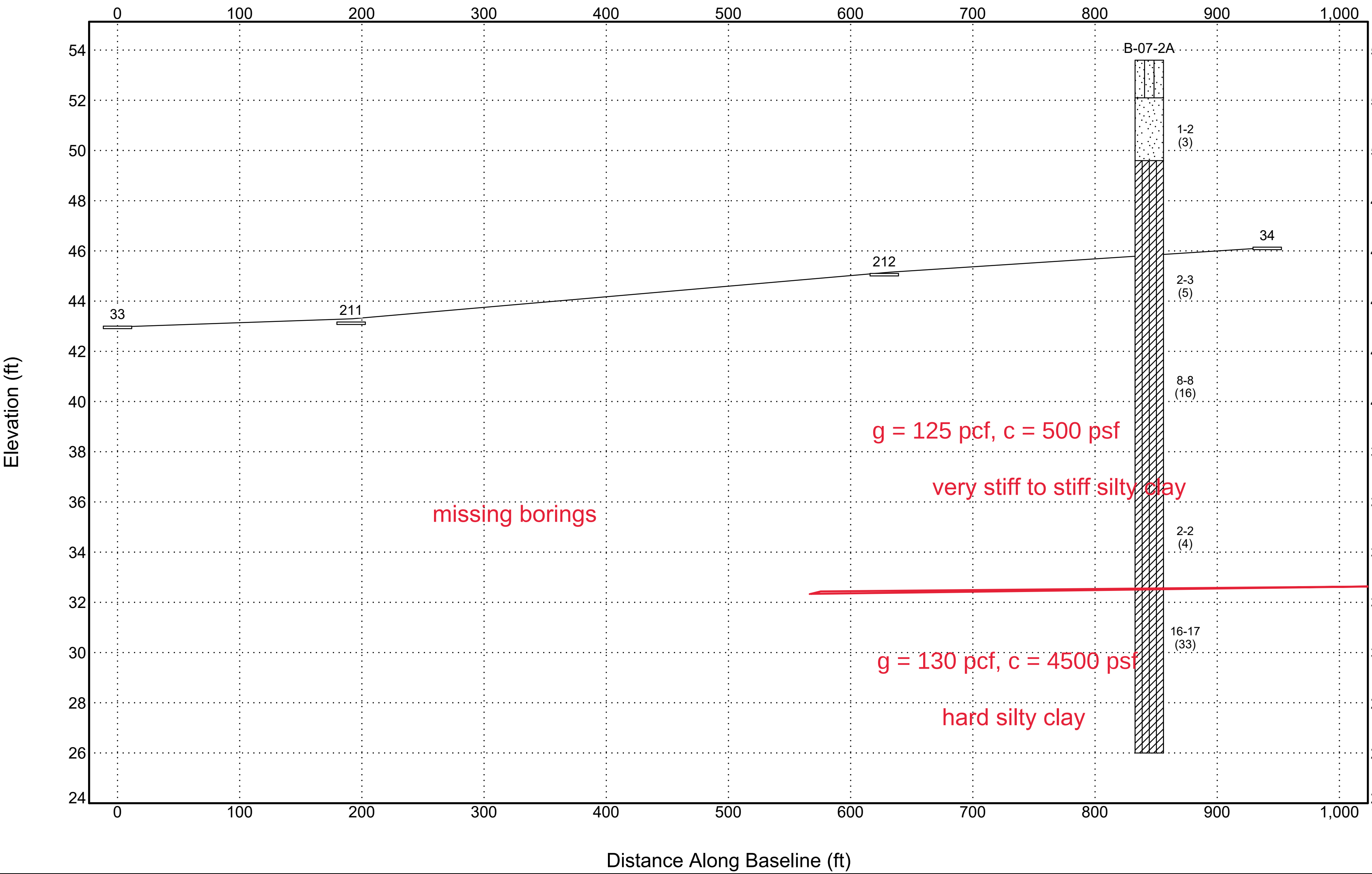
SUBSURFACE DIAGRAM
Subreach 5.3 (125 ft offset)

CLIENT ______

PROJECT NUMBER ______

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______



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Chicago District
231 S Lasalle
Chicago, IL 60604

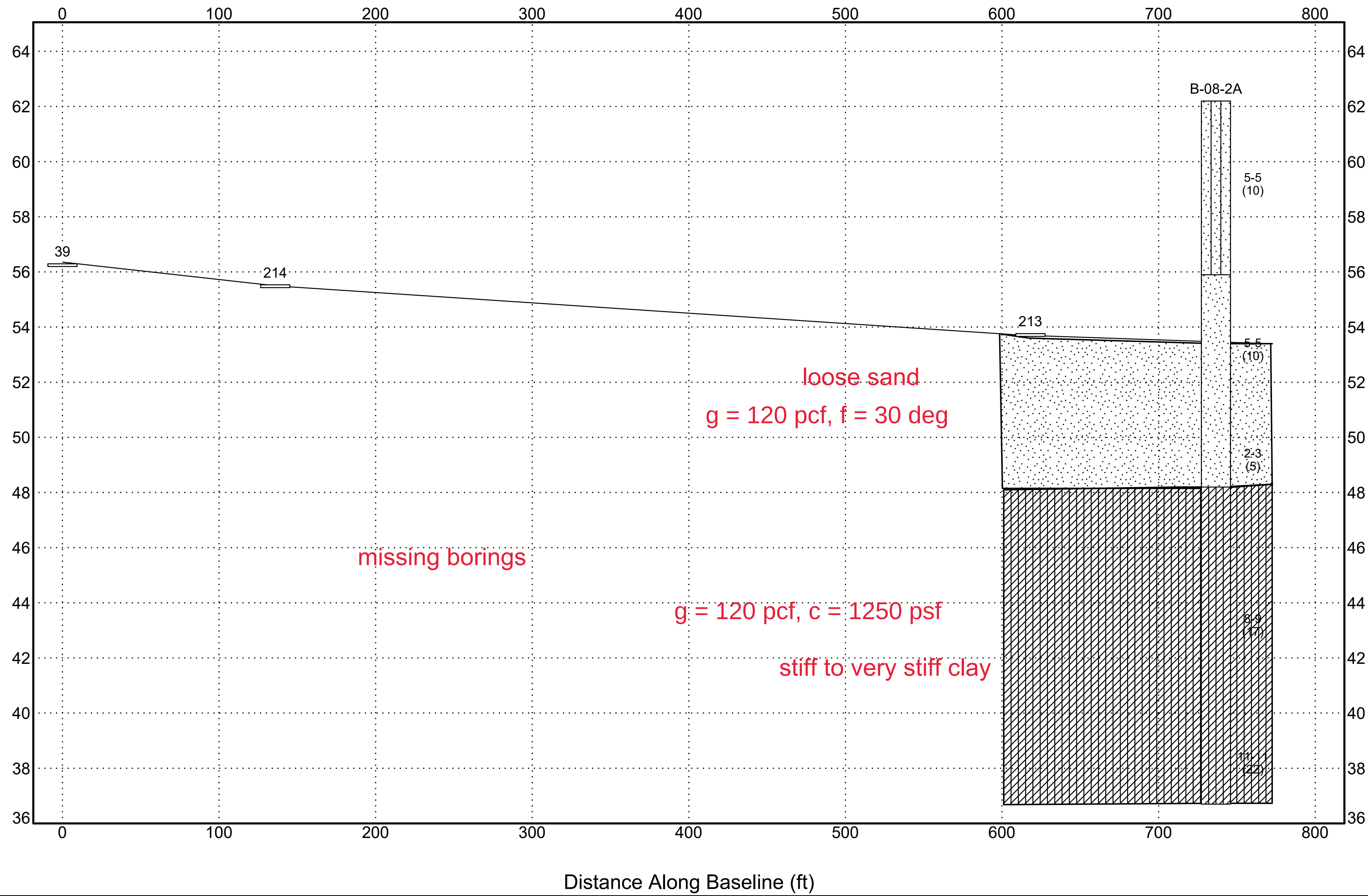
SUBSURFACE DIAGRAM
Subreach 5.1

CLIENT ______

PROJECT NUMBER ______

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______



Chicago District
231 S Lasalle
Chicago, IL 60604

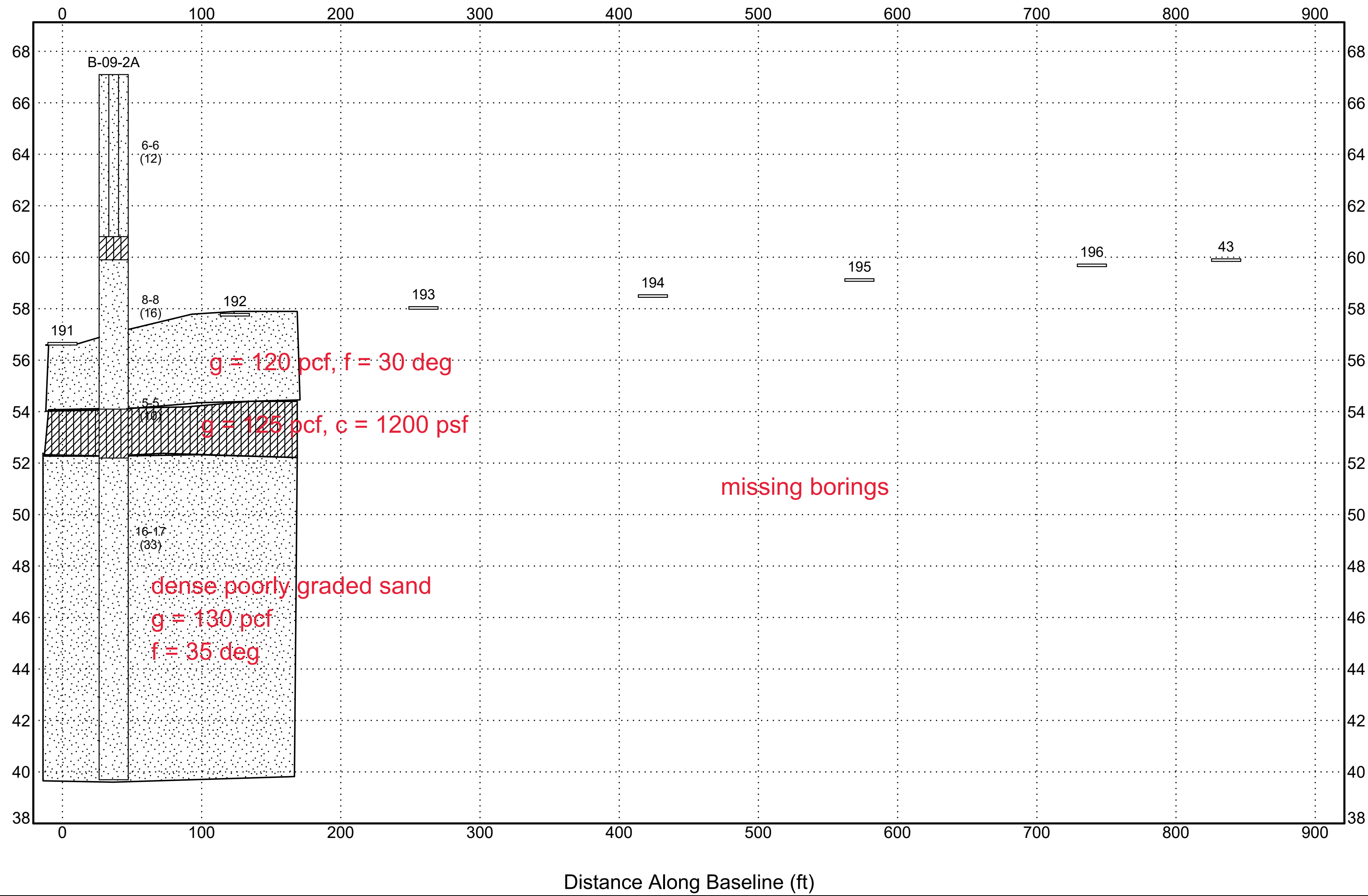
SUBSURFACE DIAGRAM
Subreach 6.1 (125 ft offset)

CLIENT ______________________

PROJECT NUMBER ______________________

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______________________



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US Army Corps of Engineers

Chicago District
231 S Lasalle
Chicago, IL 60604

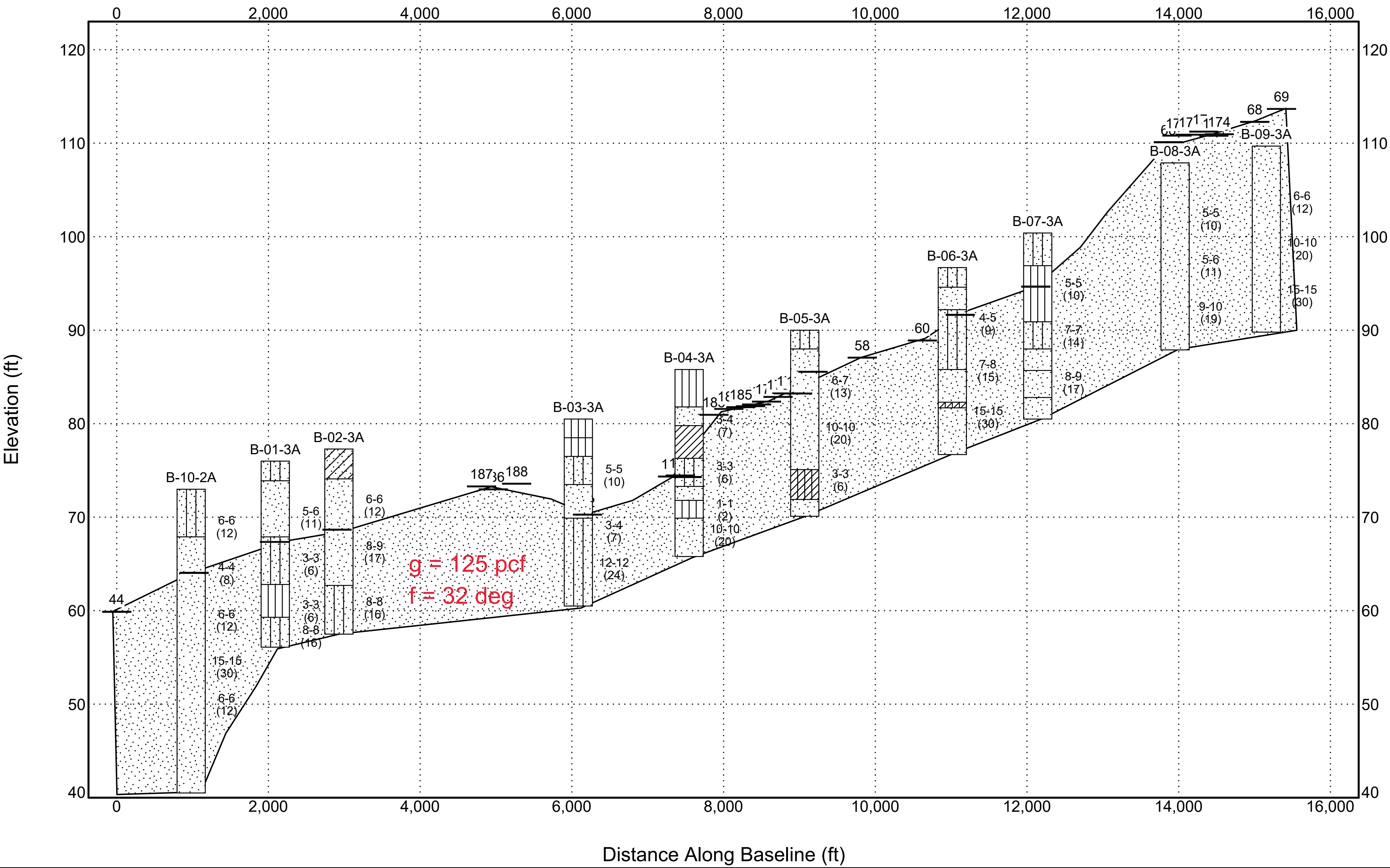
SUBSURFACE DIAGRAM
Subreach 7.1 to 10.1 (125 ft offset)

CLIENT ______________________

PROJECT NUMBER ______________________

PROJECT NAME Westminster East Garden Grove, Channel C05

PROJECT LOCATION ______________________



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SUBSURFACE DIAGRAM
Subreach 13.3 to 13.1 (offset 125 ft)

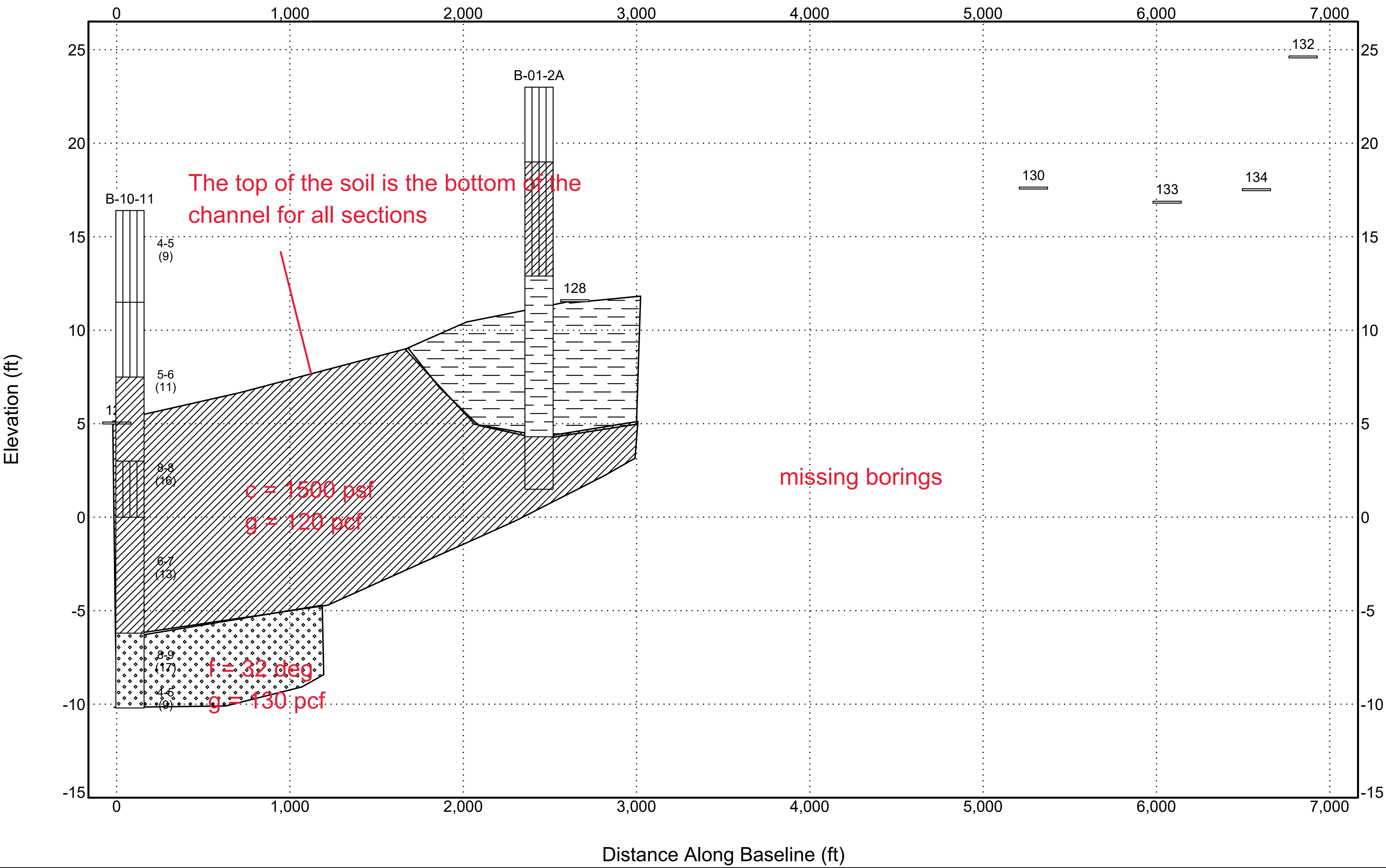
Chicago District
231 S Lasalle
Chicago, IL 60604

CLIENT ______

PROJECT NUMBER ______

PROJECT NAME Westminster Ocean View, Channel C06

PROJECT LOCATION ______



Chicago District
231 S Lasalle
Chicago, IL 60604

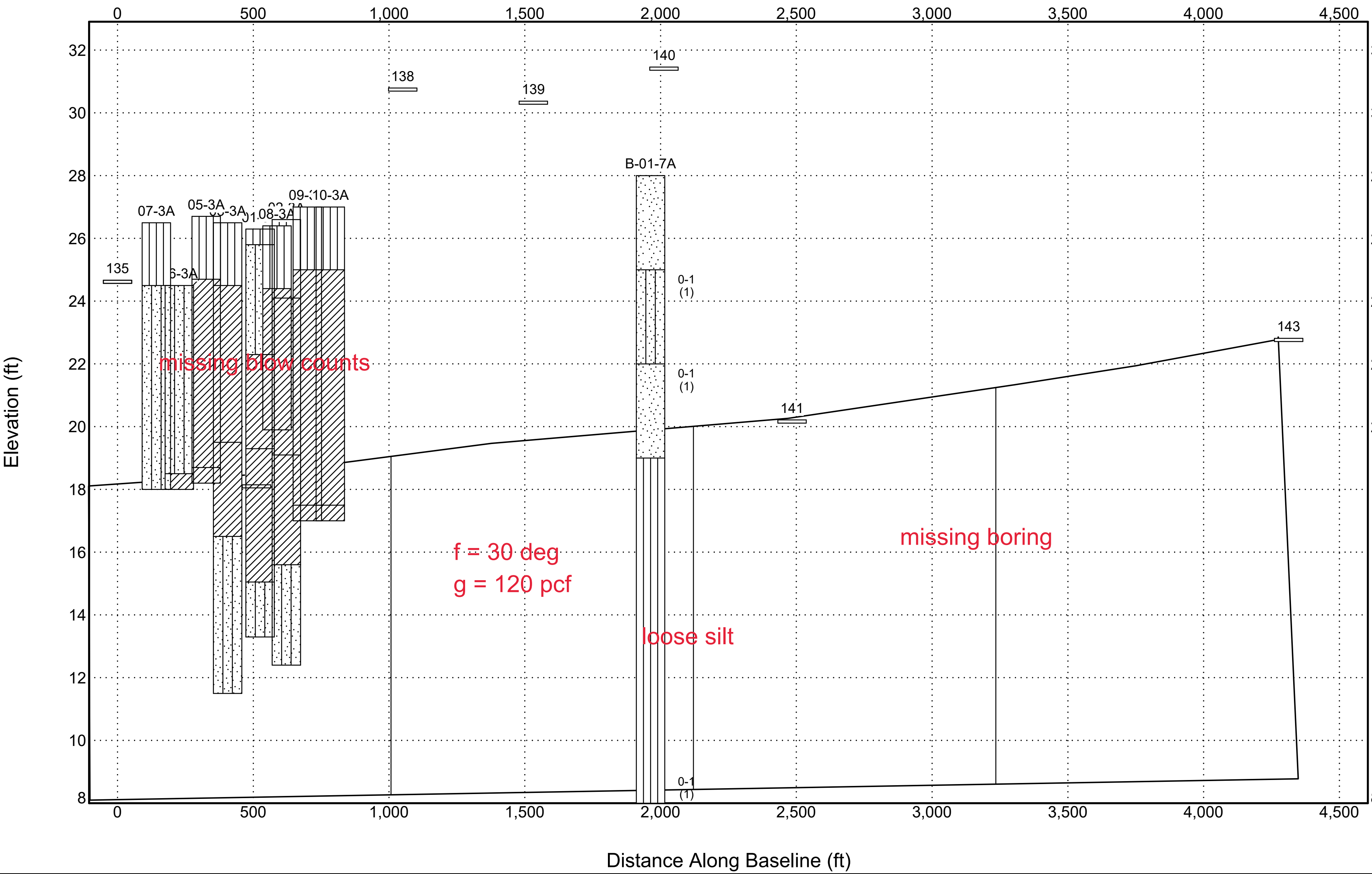
SUBSURFACE DIAGRAM
Subreach 14.2 to 16.1 (offset 125 ft)

CLIENT ______________________

PROJECT NUMBER ______________________

PROJECT NAME Westminster Ocean View, Channel C06

PROJECT LOCATION ______________________



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Bolsa Chica Ecological Reserve Overflow Analysis

BLUF

Hydraulic analyses showed minimal increase in stage due to overflow from Channel C05 into the Bolsa Chica Ecological Reserve. Therefore the analysis for stability of the levees for seepage and stability are not presently considered necessary. However, they were developed in the event that the water stage in the Bolsa Chicago Ecological Reserve is later determined to increase significantly because of this project.

Analyses

For both seepage and stability analyses, no available geotechnical data was available in the immediate area of the Bolsa Chica Ecological Reserve area. Therefore the levee properties were assumed based on knowledge that the soils in the area consist mainly of sands near the surface. Reprentative properties of sand, namely a unit weight of 110 pcf, friction angle of 30 deg and permeability of 3.28e-4 ft/s were chosen.

Though seepage and stability analyses can depend on subtle geologic factors such as the presense of a low permeability layer on the downstream side leading to sand boils, or a weak stratum leading to a slope stability failure, none of these data were available at the time of this analysis.

The surface elevations were taken from available LIDAR data. Water elevations were estimated to be normal (+ 1.9 ft NAVD88), ~50% (+ 6.5 ft NAVD 88), and at the ~crest (+11.7 ft) The seepage analyses were run then used as input for the stability analyses. The stability analyses were run using the entry exit method. The seepage boundaries were run as total head on the upstream side representing the water elevation on the upstream side and a seepage face on the downstream side.

Seepage

Seepage gradients were calculated over a roughly 5 ft distance and compared with those that led to piping as developed by Schmertmann (2000) in Figure 1. For a coefficient of uniformity of 2, which is consistent with beach sand, the critical gradient is 0.35. The results of these analyses are attached and generally show that gradients of 0.35 will not be found even if the water reaches the levee crest.

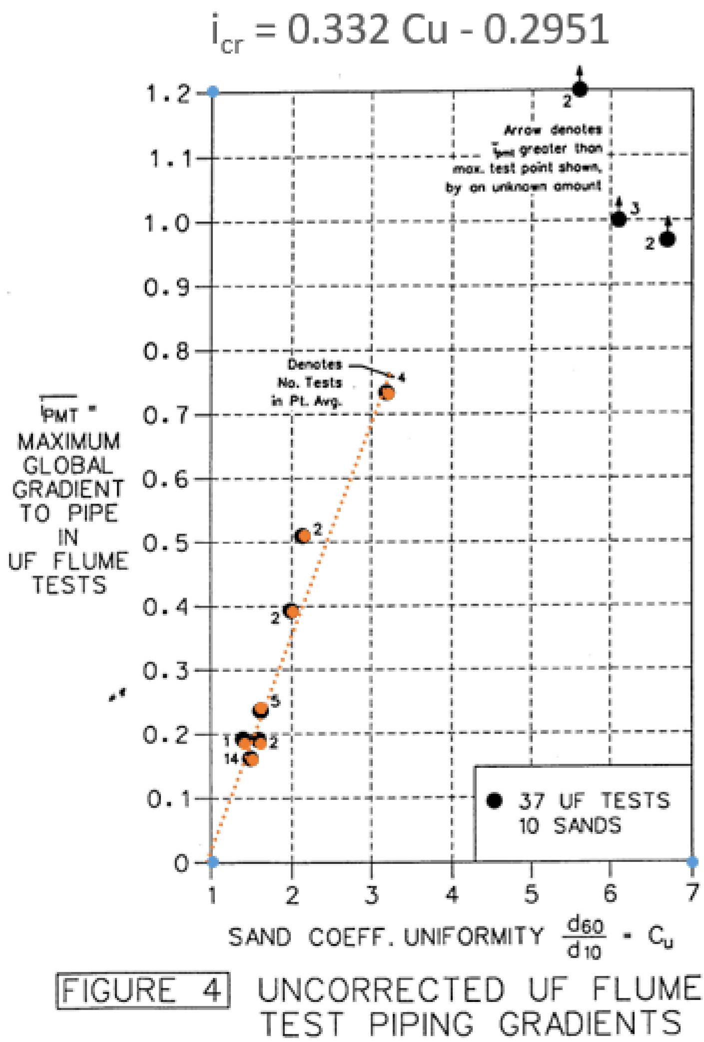
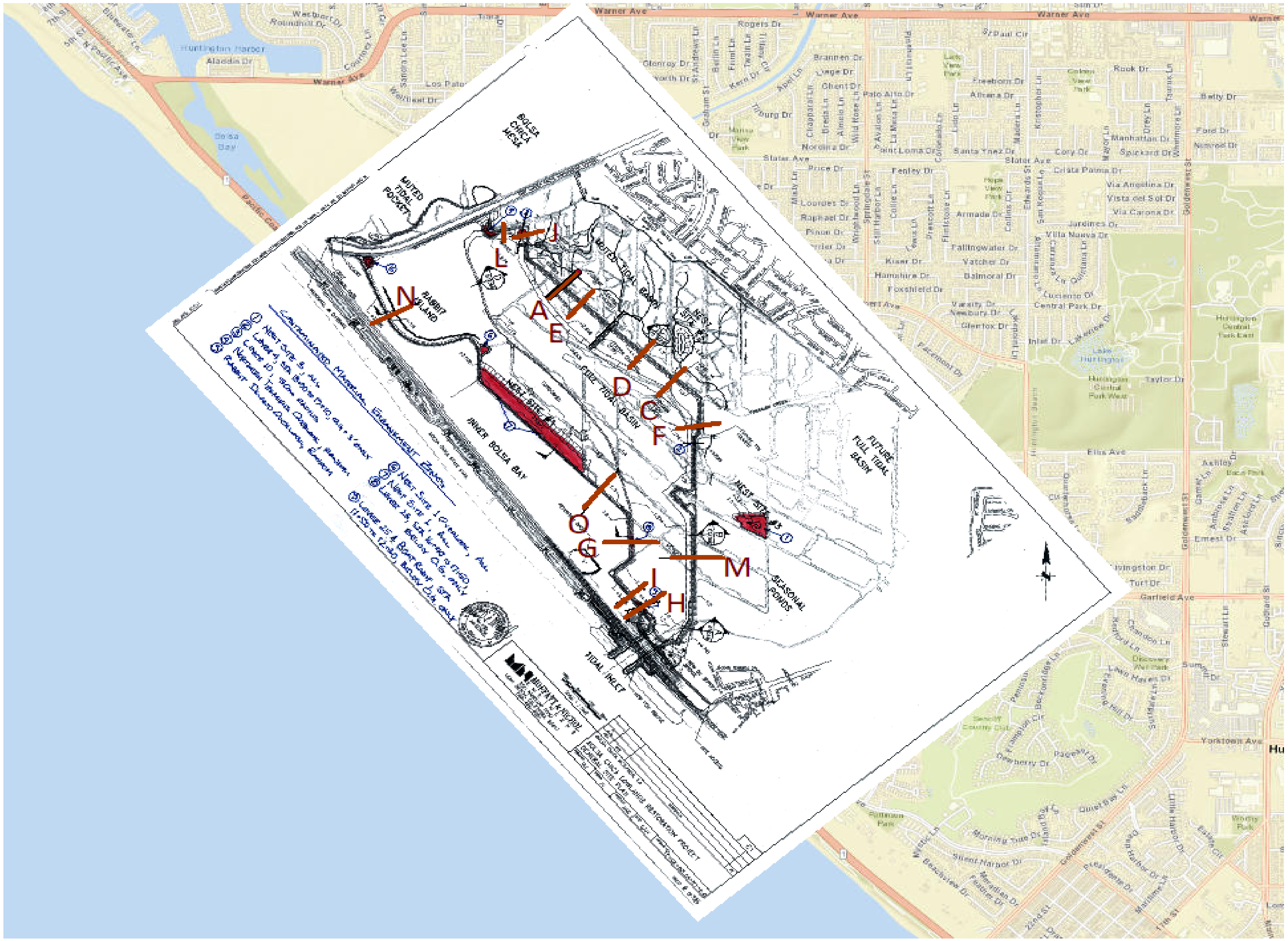


Figure 1. Critical gradient versus coefficient of uniformity (Schmertmann, 2000)

Stability

For long term the allowable factor of safety for levees is 1.5. Because the levees are assumed to be sand, the long term seepage condition is considered likely. Therefore the water elevations are compared against a factor of safety of 1.5. For the sections considered, this corresponded to an elevation of +5 ft However, given that the stages are not expected to increase more than perhaps a 0.1 ft due to the overflow from C05, stability is not considered to be a concern.

The sections chosen and the stability results are attached.


Warner Ave
Huntington Harbor
Bolsa Bay
BOLSA CHICA MESA
MUTED TIDAL POCKET
RABBIT ISLAND
INNER BOLSA BAY
NEST SITE 1
FULL TIDAL BASIN
FUTURE FULL TIDAL BASIN
SEASONAL PONDS
TIDAL INLET
J
L
N
A
E
D
C
F
O
G
M
I
H
Slater Ave
Talbert Ave
Garfield Ave
Yorktown Ave
Goldenwest St
Lake Huntington
Huntington Central Park East
Huntington Central Park West

Computed By: SP 6/28/2019
Checked By: JWS 9/10/2019

dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2+dy^2)
5.9	4.76	34.81	22.6576	57.4676	7.58073875

283	11
305	2.9
dY	dX
8.1	22

2.716049

X	Y	H
300.2929	3.7229	1.398704
294.8072	1.712453	1.472662

dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2+dy^2)
5.48566	2.010447	30.09247	4.041897	34.13436	5.842462

DH	DL	i
0.073958	5.842462	0.012659

0.021381051
0.174341848

X	Y	H
299.4656	3.945333	3.945333
294.1702	-0.06303	5.696935

dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2+dy^2)
5.2954	4.008367	28.04126	16.06701	44.108268	6.641405572

DH	DL	i
1.7516017	6.641405572	0.26374

0.024051

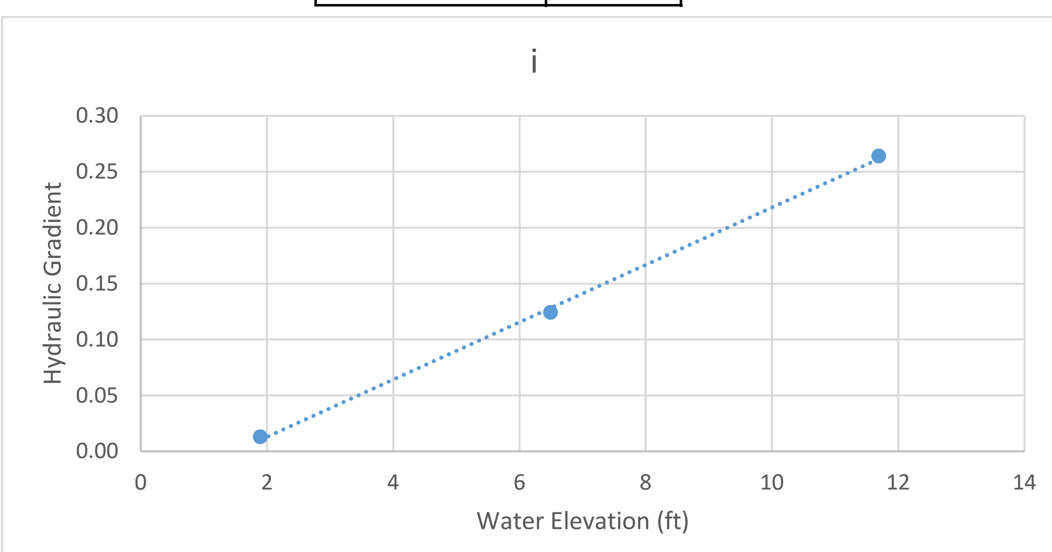
Section A

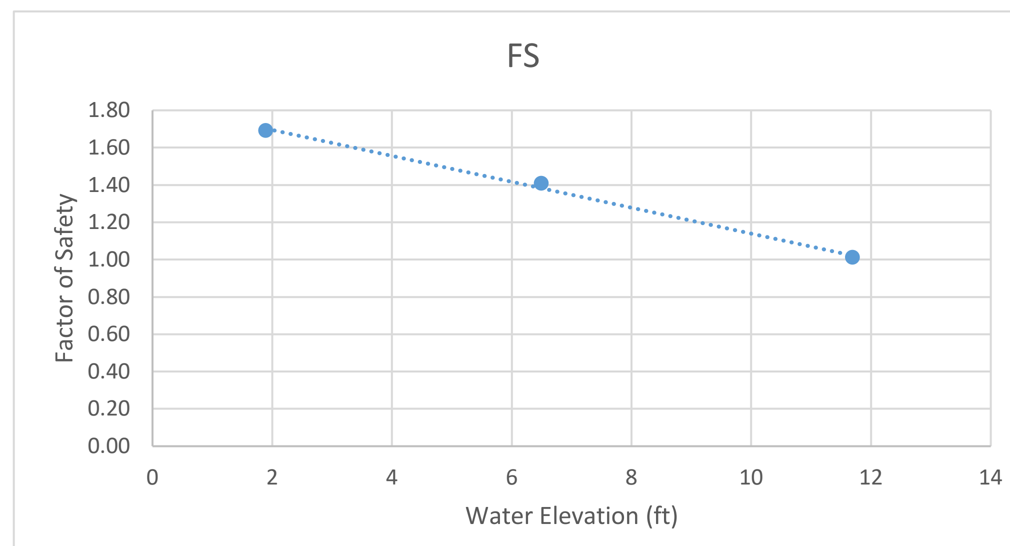
Seepage

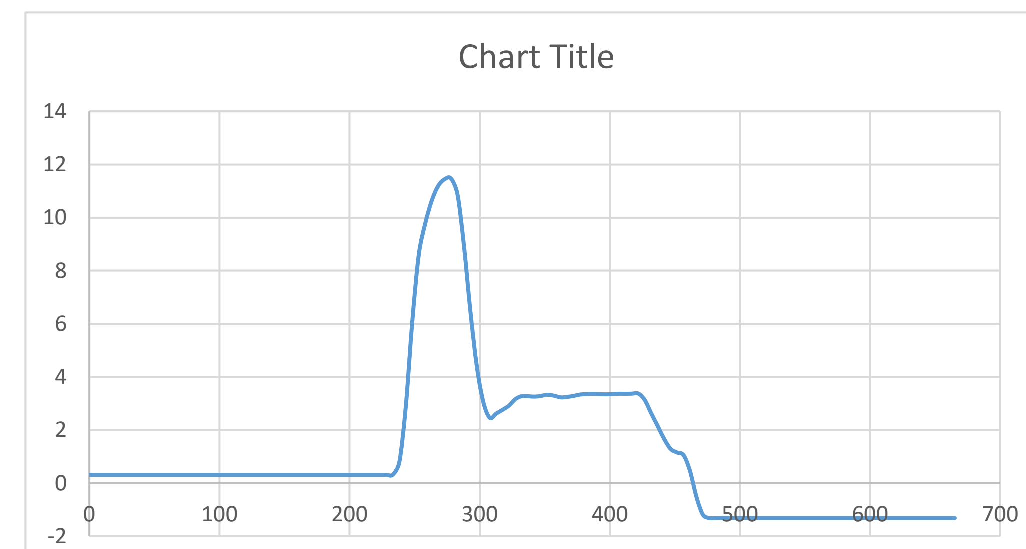
Water elevation	i
1.9	0.01
6.5	0.12
11.7	0.26

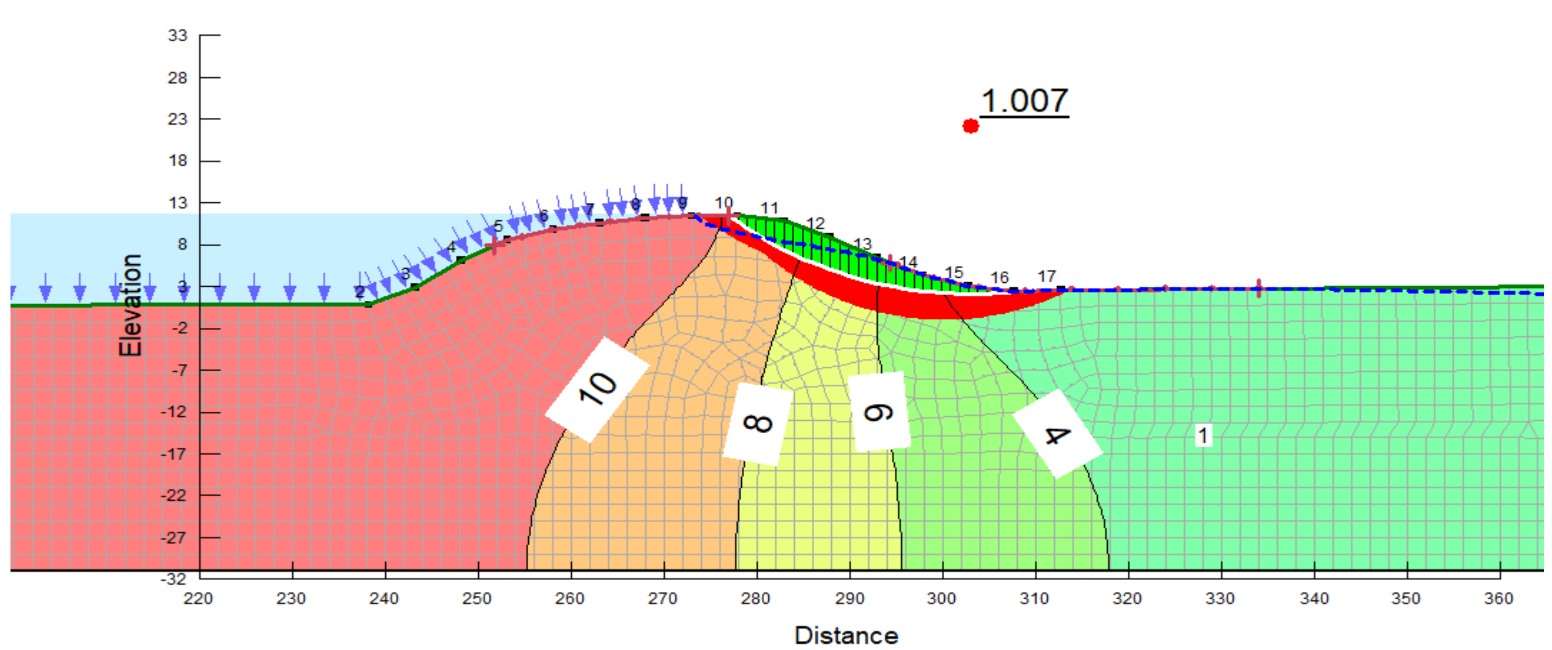
Slope Stability

Water elevation	FS
1.9	1.69
6.5	1.41
11.7	1.01









Computed By: SP 6/28/2019
Checked By: JWS 9/10/2019

OverflowC

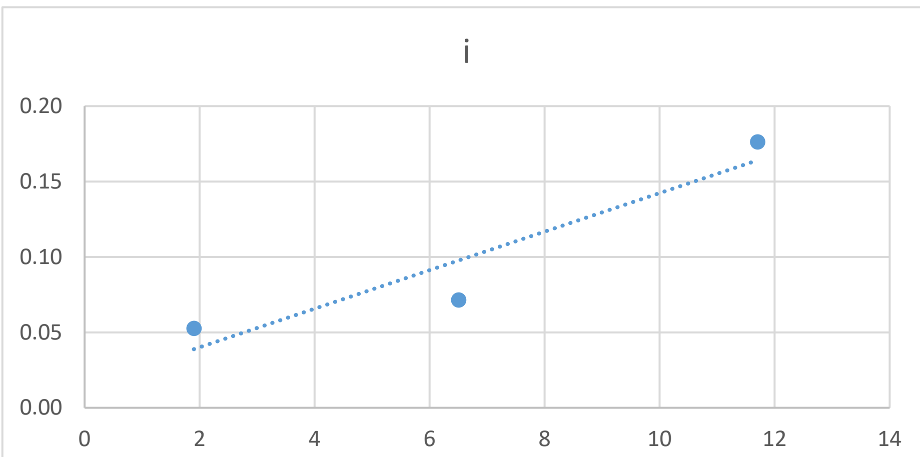
Water El	X	Y	H	dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2	dH	dL	i
	506.10496	6.4287	-0.234692	4.76945	1.706598	22.74765	2.91247571	25.66013	5.065583	0.266836	5.065583	0.052676
1.9	501.33551	4.7221023	0.032144									
	504.4456	7.2213	3.623912	4.6856	1.81517	21.95485	3.294842129	25.24969	5.024907	0.35957	5.024907	0.071558
6.5	499.76	5.40613	3.983482									
	507.76432	5.6361	5.524135	4.8499	1.738405	23.52153	3.022051249	26.54358	5.152046	0.908356	5.152046	0.17631
11.7	502.91442	3.8976952	6.432491									

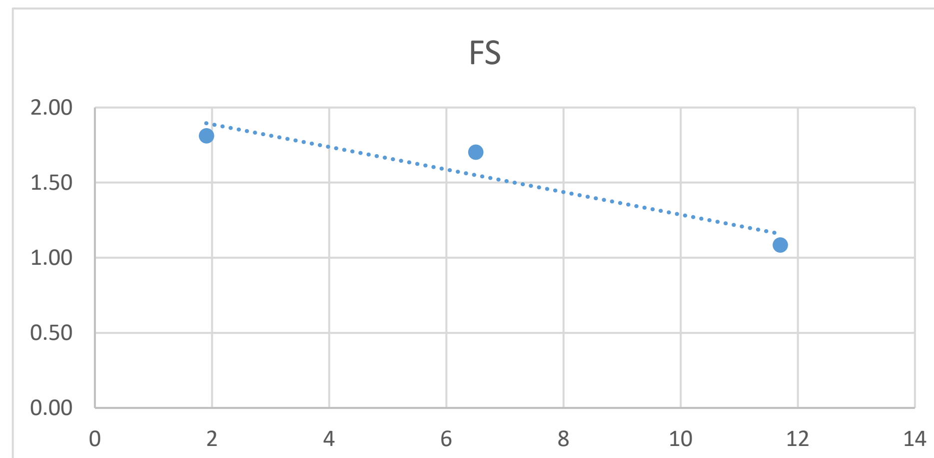
Seepage

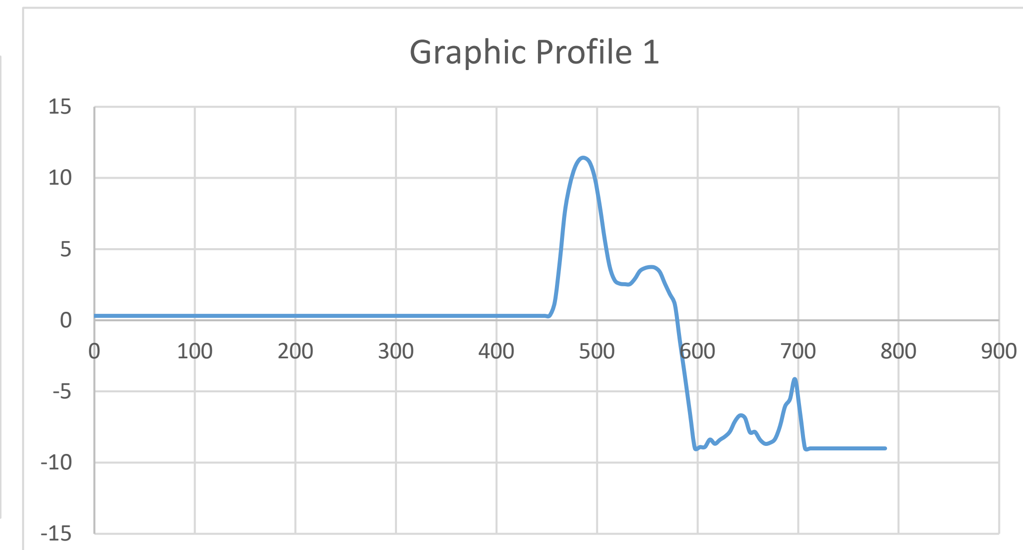
Water Elevation	FS
1.9	1.81
6.5	1.71
11.7	1.09

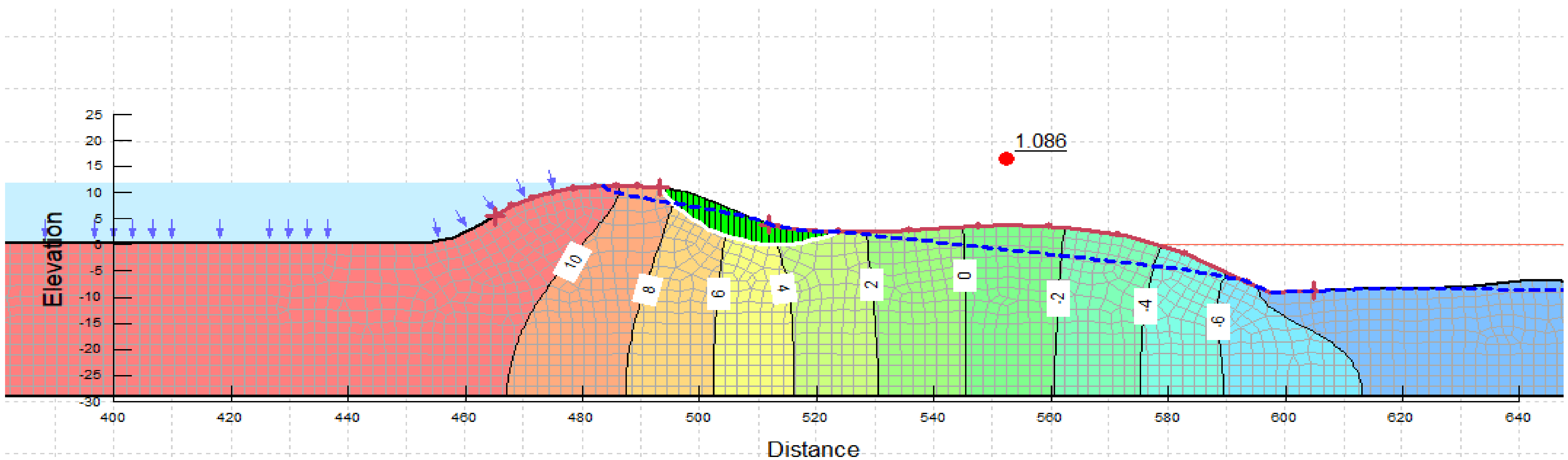
Slope Stability

Water Elevation	i
1.9	0.05
6.5	0.07
11.7	0.18









Computed By: SP 6/28/2019
Checked By: JWS 9/10/2019

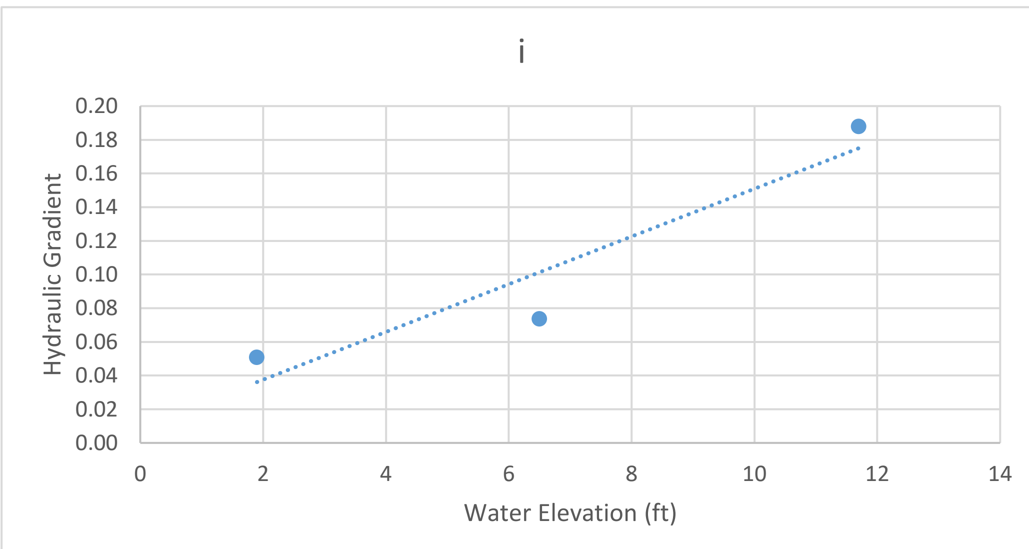
Water El	X	Y	H	dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2+dy^2)	dH	dL	i	
		618.9	4.0	-1.1	5.4	3.9	29.1	15.3	44.4	6.7	0.3	6.7	0.05
1.9		613.5	0.1	-0.7									
		618.9	4.0	2.2	5.4	3.9	29.1	15.3	44.4	6.7	0.5	6.7	0.07
6.5		613.5	0.1	2.7									
		618.9	4.0	4.0	5.4	3.9	29.1	15.3	44.4	6.7	1.3	6.7	0.19
11.7		613.5	0.1	5.2									

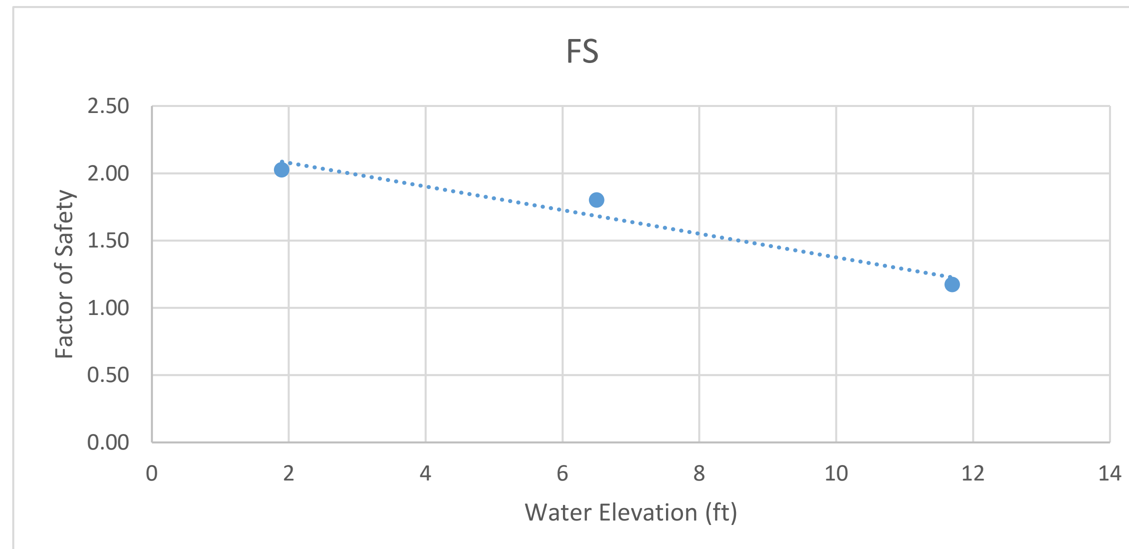
Seepage

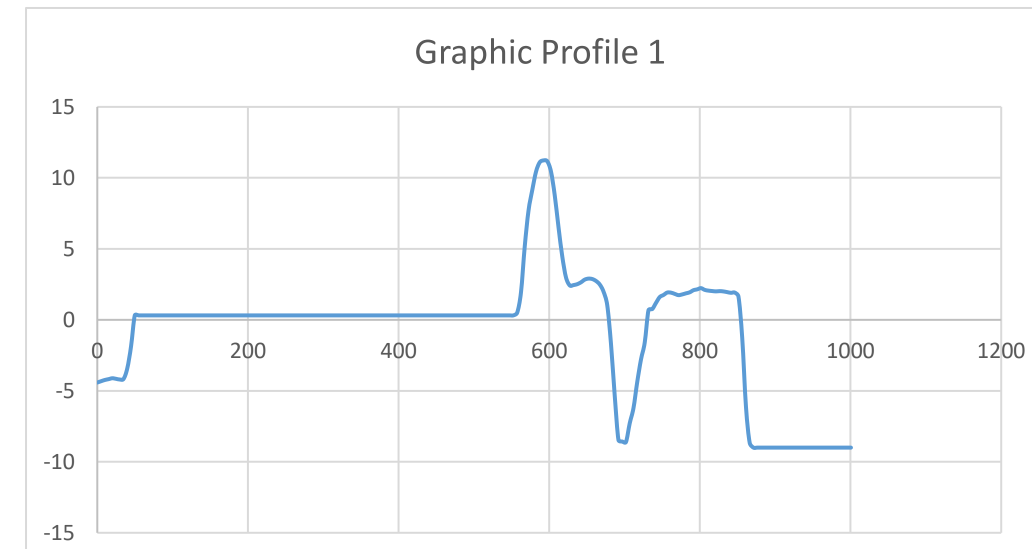
Water Elevat	i
1.9	**0.05**
6.5	**0.07**
11.7	**0.19**

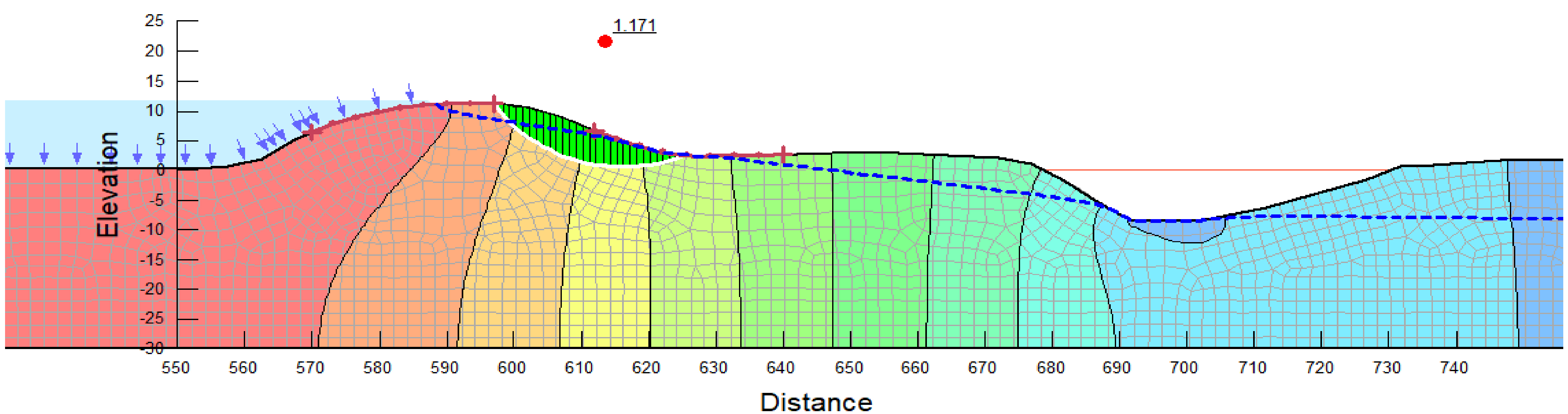
Slope Stability

Water Elev	FS
1.9	**2.02**
6.5	**1.80**
11.7	**1.17**









Computed By: SP 6/28/2019
Checked By: JWS 9/10/2019

OverflowE

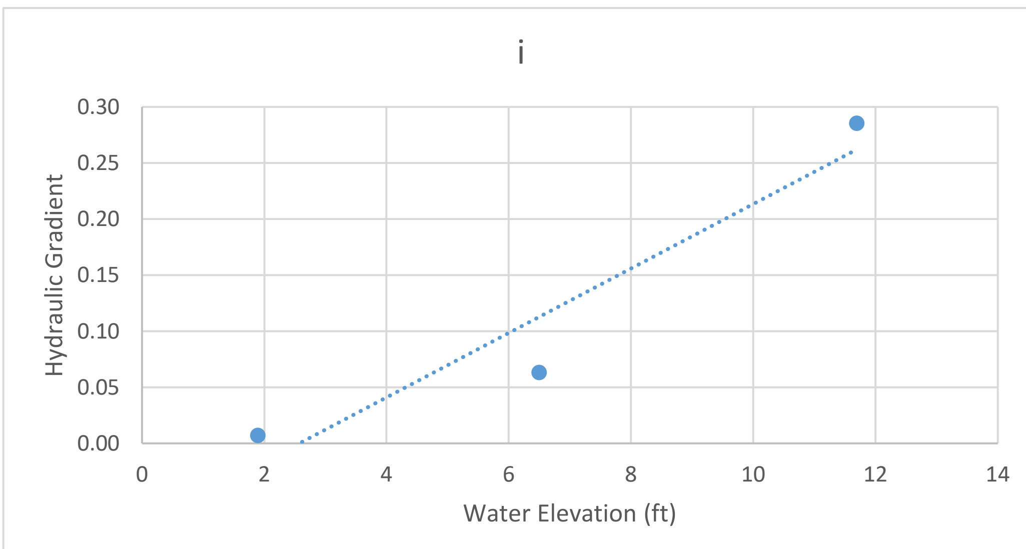
Water El	X	Y	H	dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2+dy^2)	dH	dL	i
	322.88275	4.3262	1.559914	5.44835	3.686992	29.68452	13.59391	43.27843	6.578634224	0.045992	6.578634	0.006991
1.9	317.4344	0.6392	1.605906									
	322.88275	4.3262	4.110492	5.44835	3.686992	29.68452	13.59391	43.27843	6.578634224	0.415749	6.578634	0.063197
6.5	317.4344	0.6392	4.526241									
	322.88275	4.3262	4.3262	5.44835	3.686992	29.68452	13.59391	43.27843	6.578634224	1.878674	6.578634	0.285572
11.7	317.4344	0.6392	6.204874									

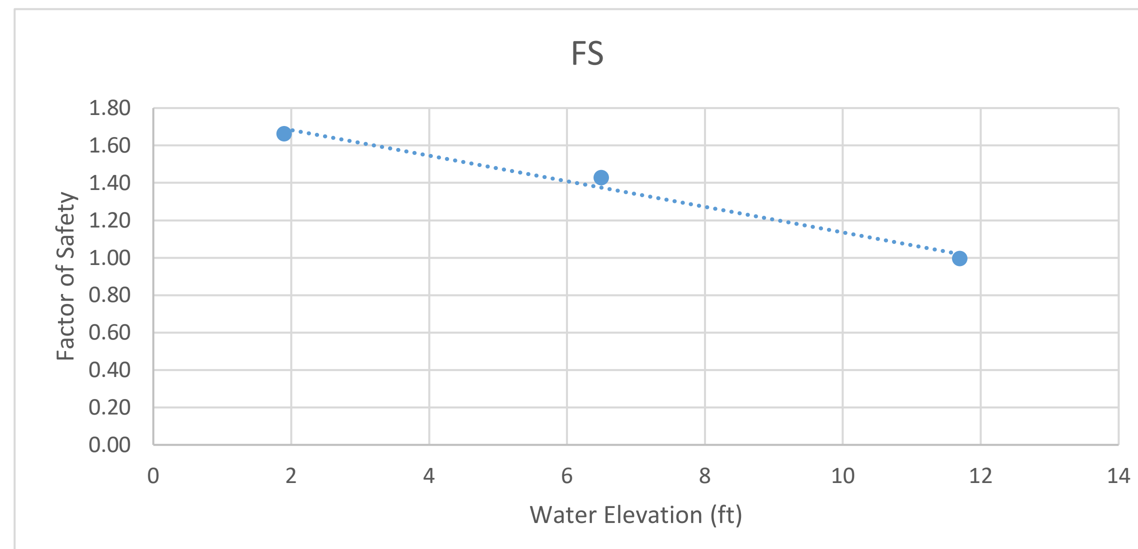
Seepage

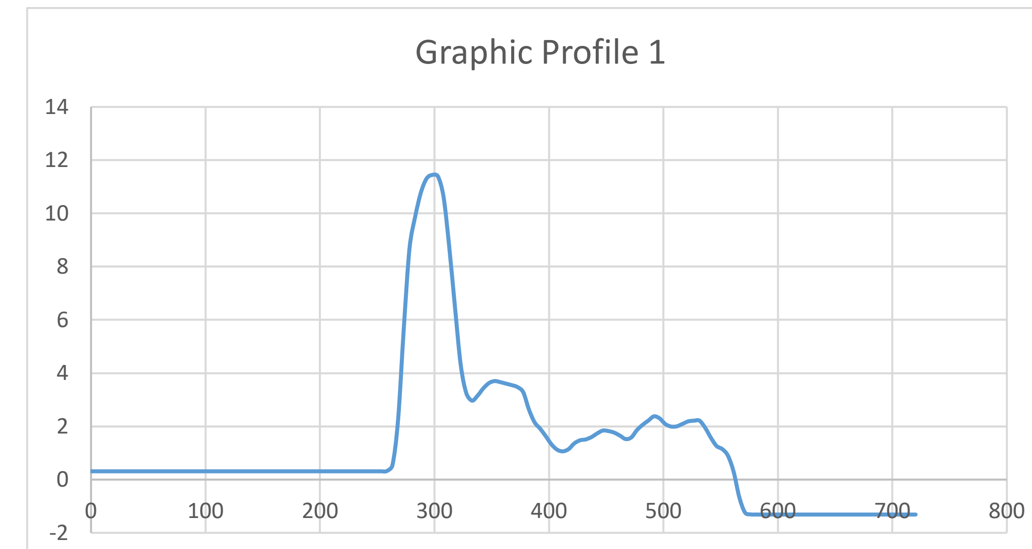
Water Elevation	i
1.9	0.01
6.5	0.06
11.7	0.29

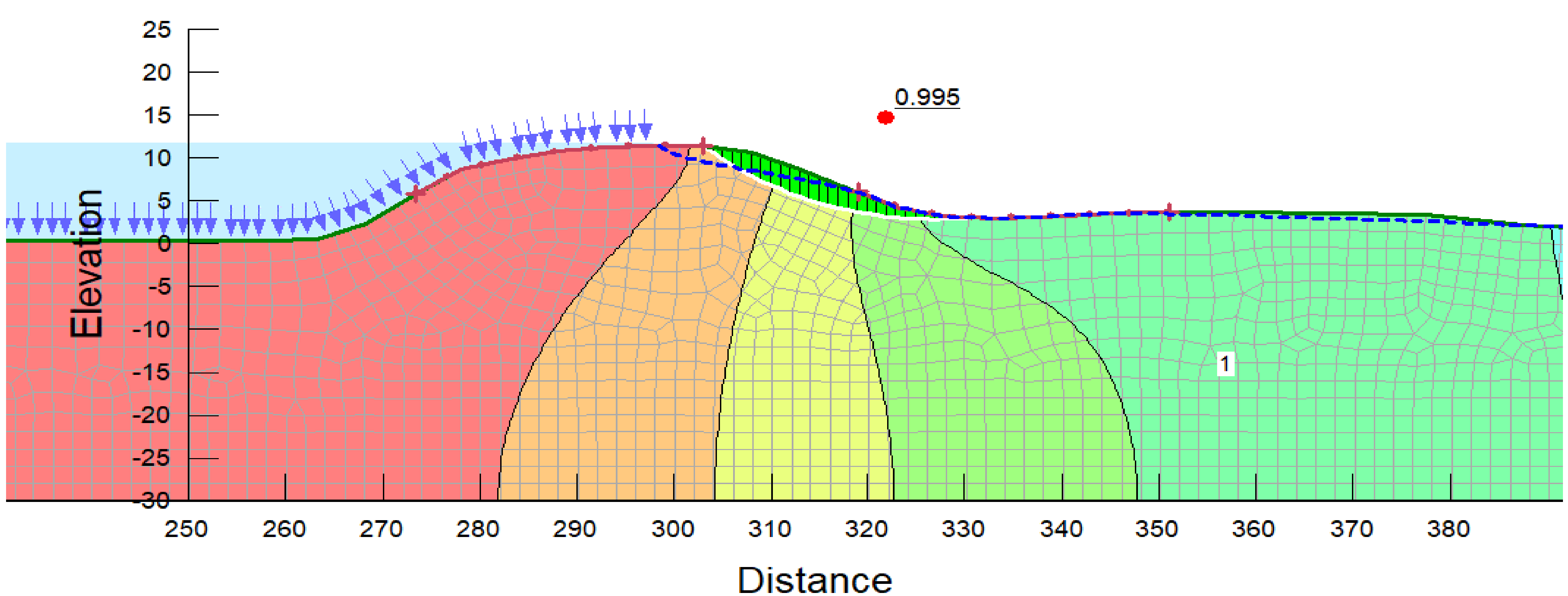
Slope Stability

Water Elevation	FS
1.9	1.66
6.5	1.43
11.7	1.00









Computed By: SP 6/28/2019
Checked By: JWS 9/10/2019

X	Y	H
233.067	4.816733	4.4988386
228.2056	0.466672	4.7281262

DH	DL	i
0.229288	7.580739	0.030246

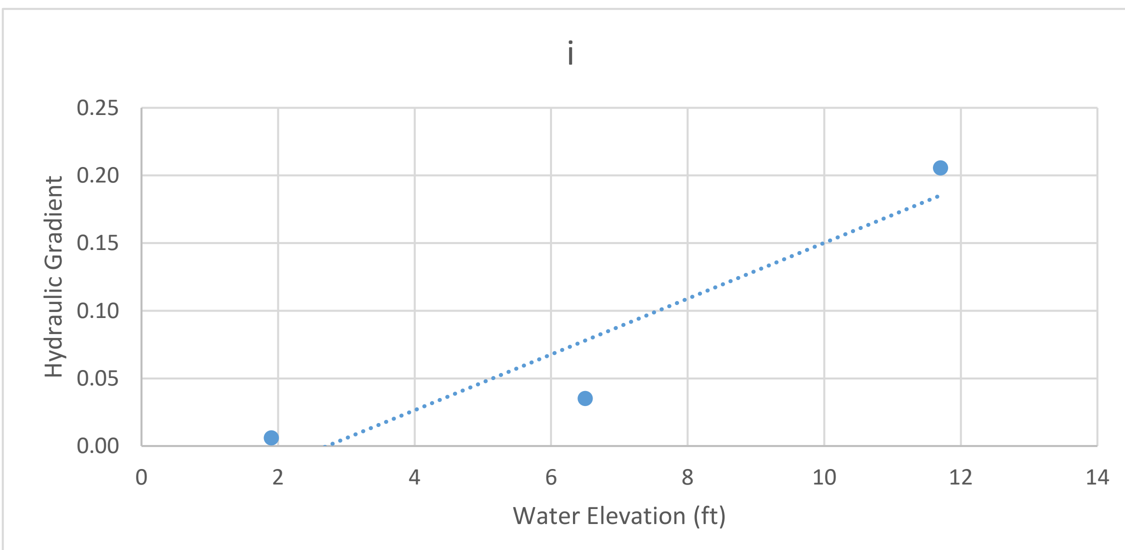
dx	dy	dx^2	dy^2	dx^2+dy^2	**SQRT(dx^2+dy^2)**
4.86135	4.350061	23.63272382	18.92303	42.55575	6.523477

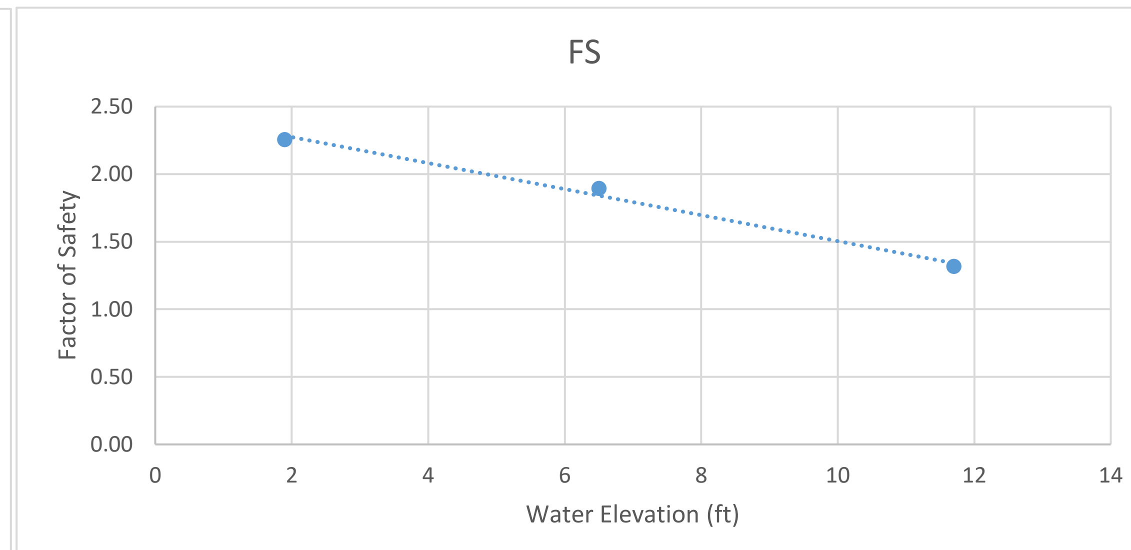
Seepage

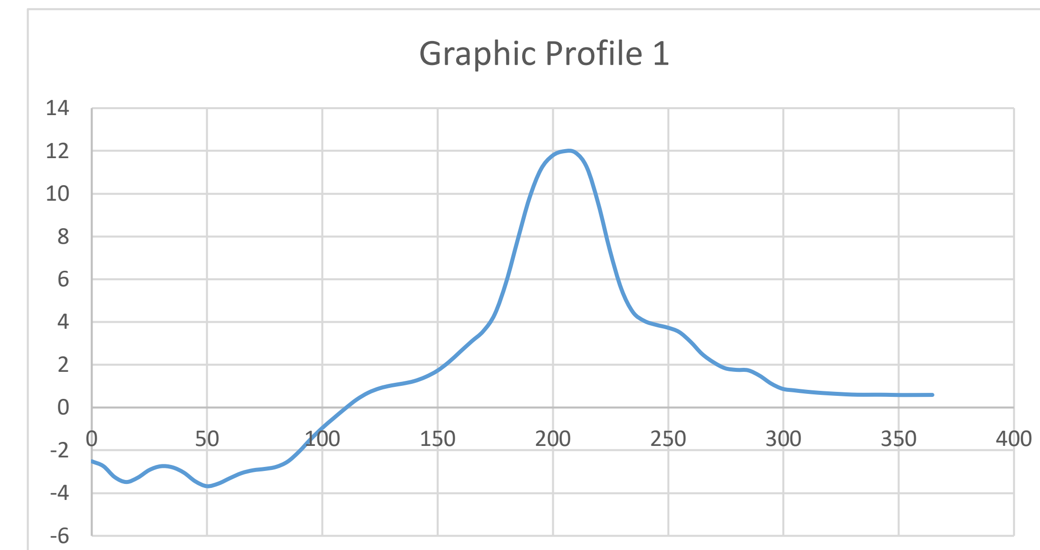
Water Elevation	i
1.9	0.01
6.5	0.04
11.7	0.21

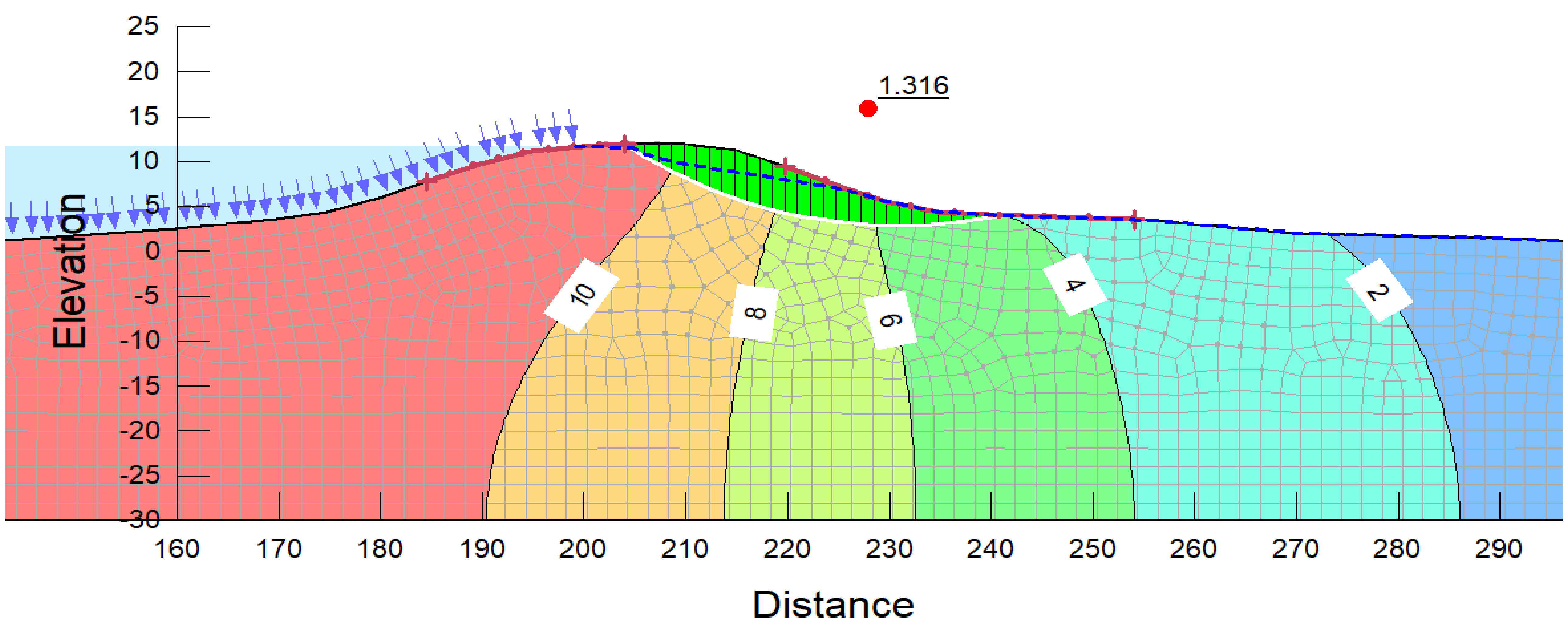
Slope Stability

Water Elevation	FS
1.9	2.26
6.5	1.89
11.7	1.32









Calculated by: SP 6/28/2019
Checked by: JWS 9/10/2019

OverflowG

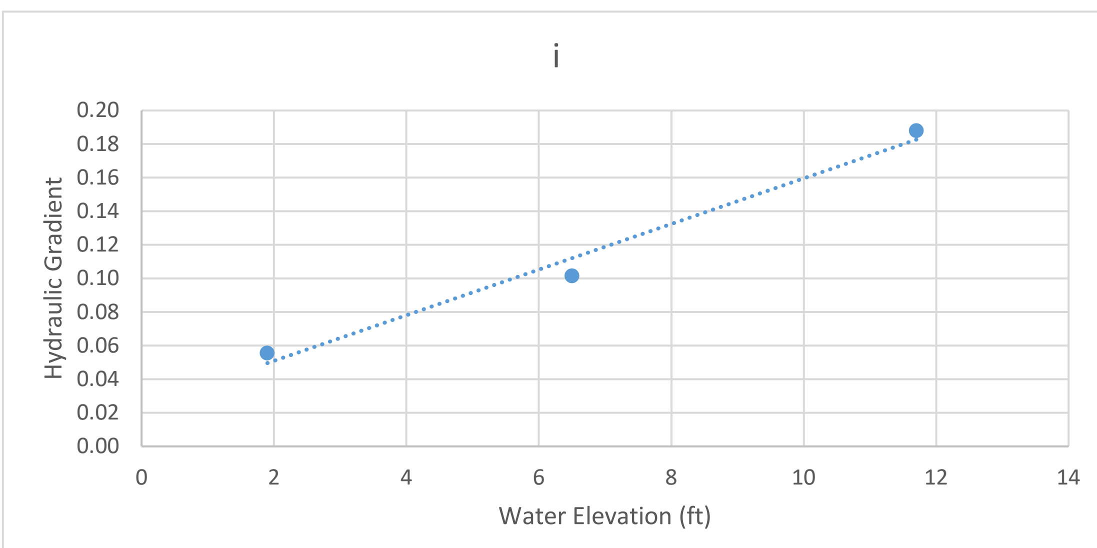
Water El	X	Y	H	dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2	dH	dL	i
	506.174	3.45126	-0.64144	5.09591	2.079109	25.9683	4.322693	30.29099	5.503725	0.30432416	5.503725	0.055294
1.9	501.078	1.3721513	-0.33711									
	506.174	3.45126	1.860541	5.09591	2.079109	25.9683	4.322693	30.29099	5.503725	0.5574951	5.503725	0.101294
6.5	501.078	1.3721513	2.418036									
	506.174	3.45126	3.45126	5.09591	2.079109	25.9683	4.322693	30.29099	5.503725	1.0332237	5.503725	0.187732
11.7	501.078	1.3721513	4.617389									

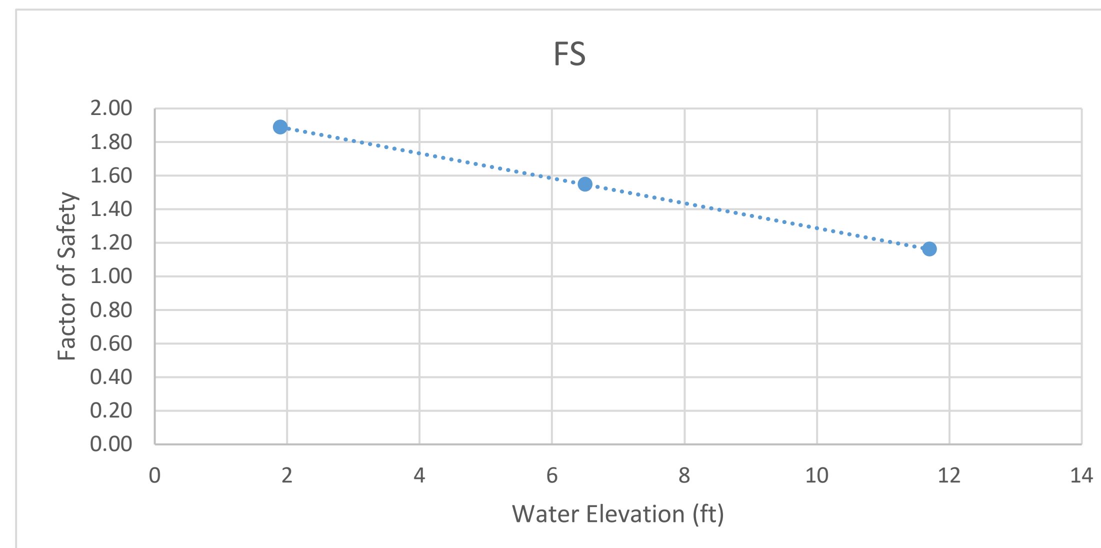
Seepage

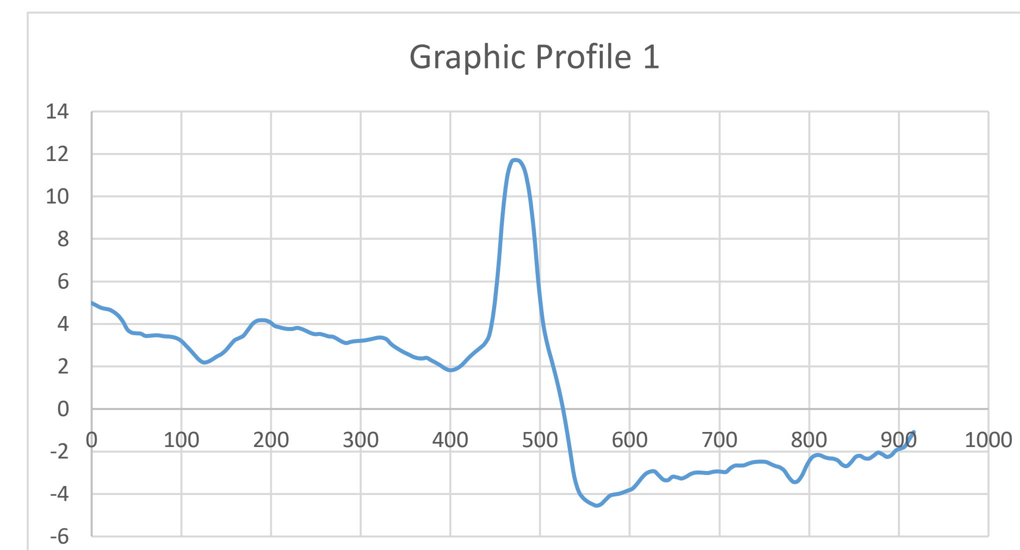
Water Elevation	i
1.9	0.06
6.5	0.10
11.7	0.19

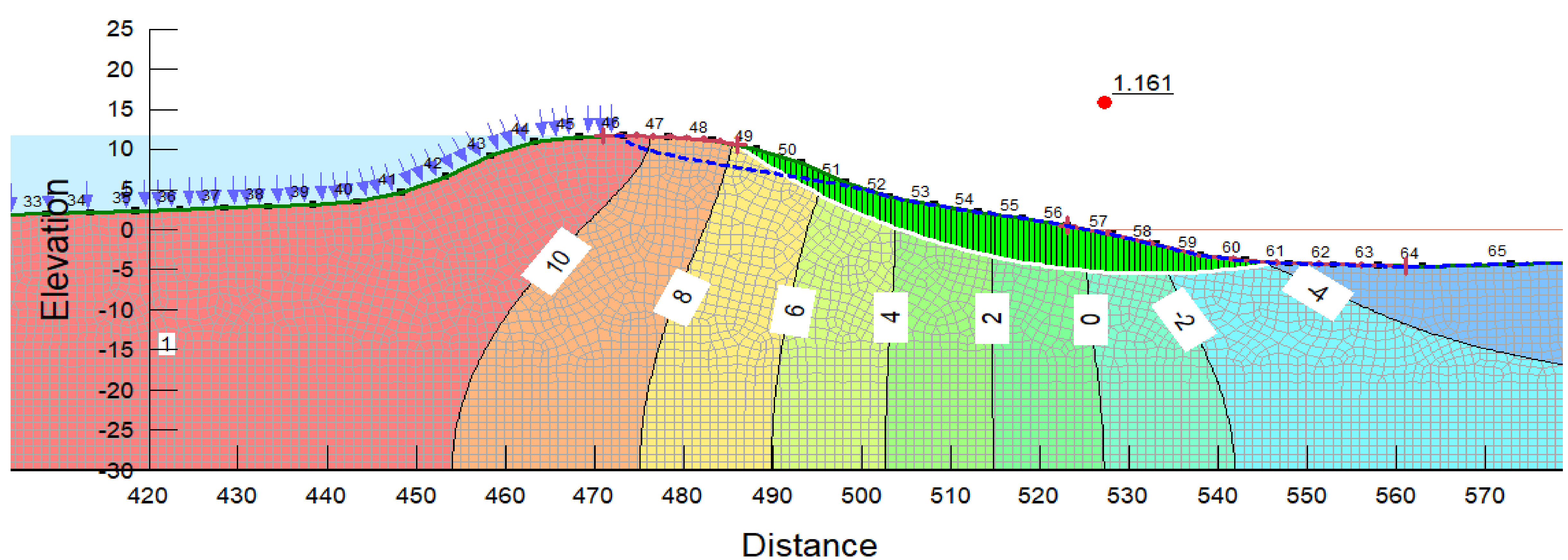
Slope Stability

Water Elevation	FS
1.9	1.89
6.5	1.55
11.7	1.16









Calculated by: SP 6/28/2019
Checked by: JWS 9/10/2019

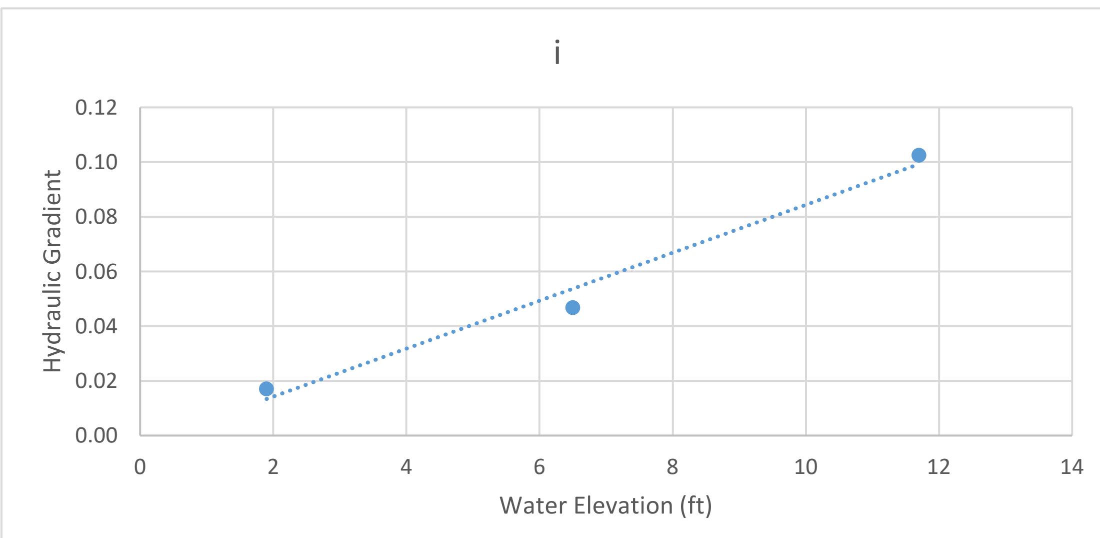
Water El	X	Y	H	dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2	dH	dL	i
	247.2418	5.8508667	0.924211	4.97022	5.020023	24.70309	25.20063	49.90371	7.064256	0.12046047	7.064256	0.017052
1.9	242.2716	0.83084408	1.044672									
	247.2418	5.8508667	3.828921	4.97022	5.020023	24.70309	25.20063	49.90371	7.064256	0.3302926	7.064256	0.046755
6.5	242.2716	0.83084408	4.159214									
	247.2418	5.8508667	5.850867	4.70125	8.199251	22.10175	67.22772	89.32947	9.451427	0.9689522	9.451427	0.102519
11.7	242.5405	-2.3483846	6.819819									

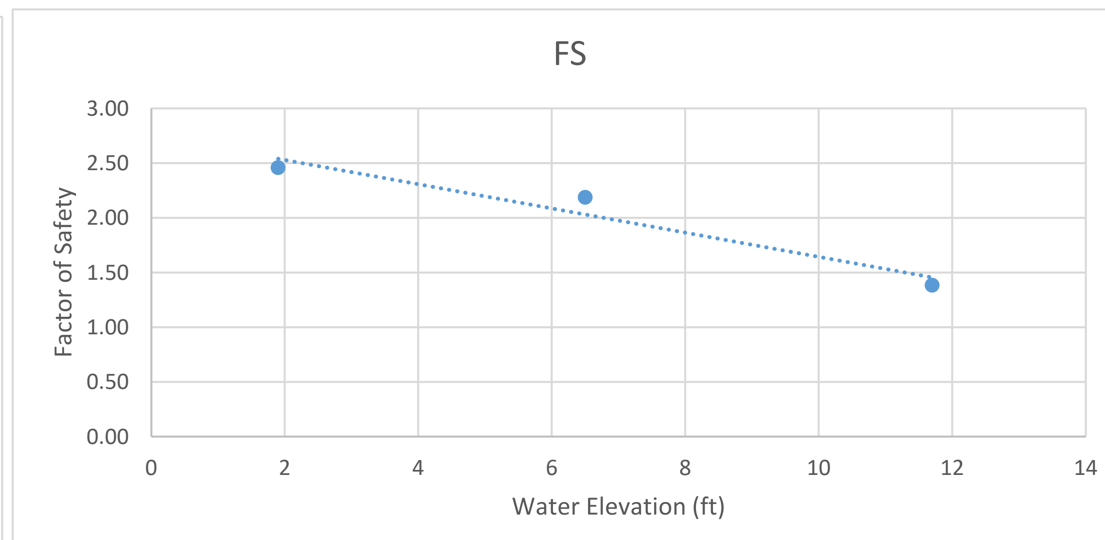
Seepage

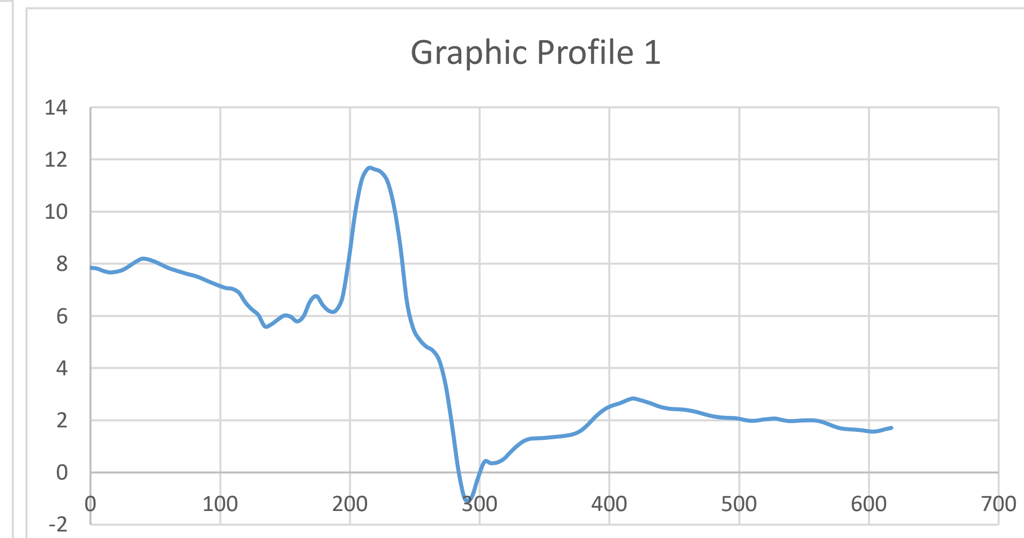
Water Elevation	i
1.9	0.02
6.5	0.05
11.7	0.10

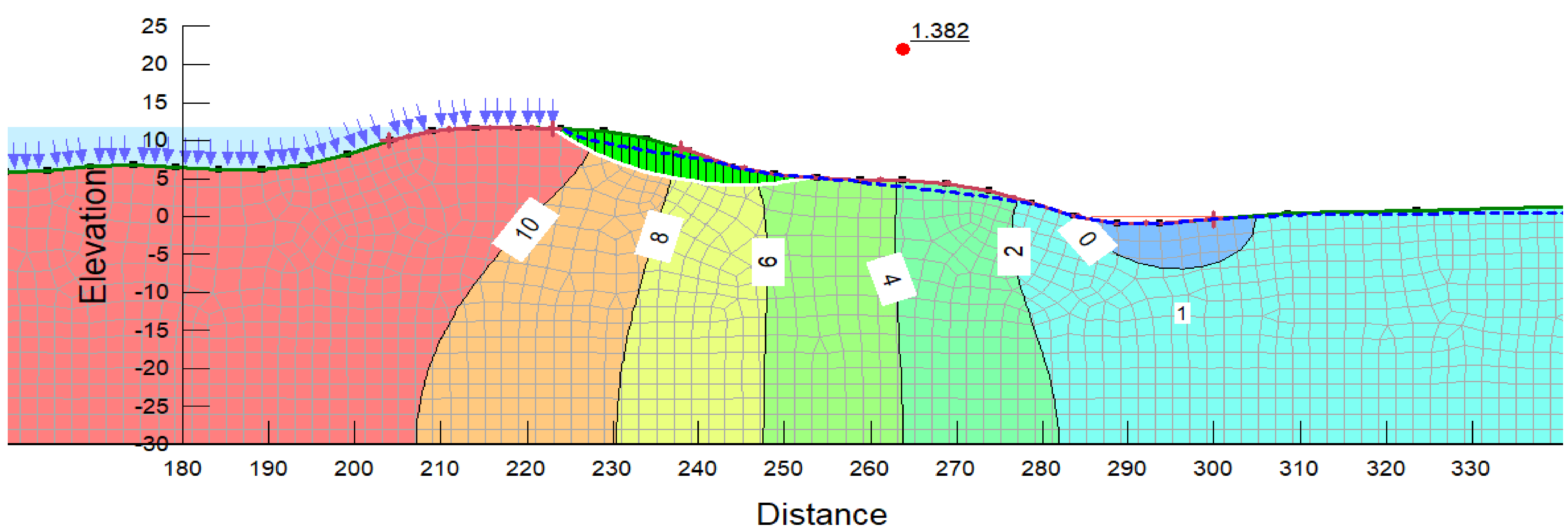
Slope Stability

Water Elevation	FS
1.9	2.46
6.5	2.19
11.7	1.38









Calculated by: SP 6/28/2019
Checked by: JWS 9/10/2019

Overflowl

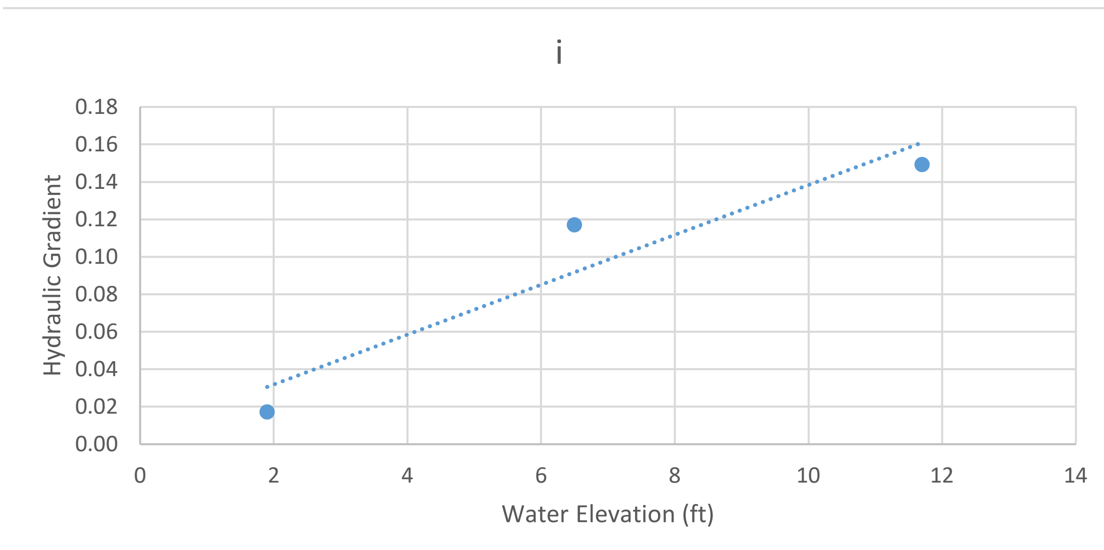
Water El	X	Y	H	dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2	dH	dL	i
	260.9057	3.594367	0.96804185	5.71741	3.572531	32.68878	12.76298	45.45176	6.741792	0.115713	6.741791835	0.017164
1.9	255.1883	0.021835	1.0837549									
	254.2585	6.7487	3.4752139	4.85554	1.68914	23.57627	2.853196	26.42946	5.140959	0.60151	5.140959471	0.117003
6.5	249.4029	5.05956	4.0767239									
	255.9203	5.932933	5.431006	5.69865	3.344855	32.47461	11.18805	43.66266	6.607773	0.985558	6.607773007	0.149151
11.7	250.2216	2.588079	6.4165642									

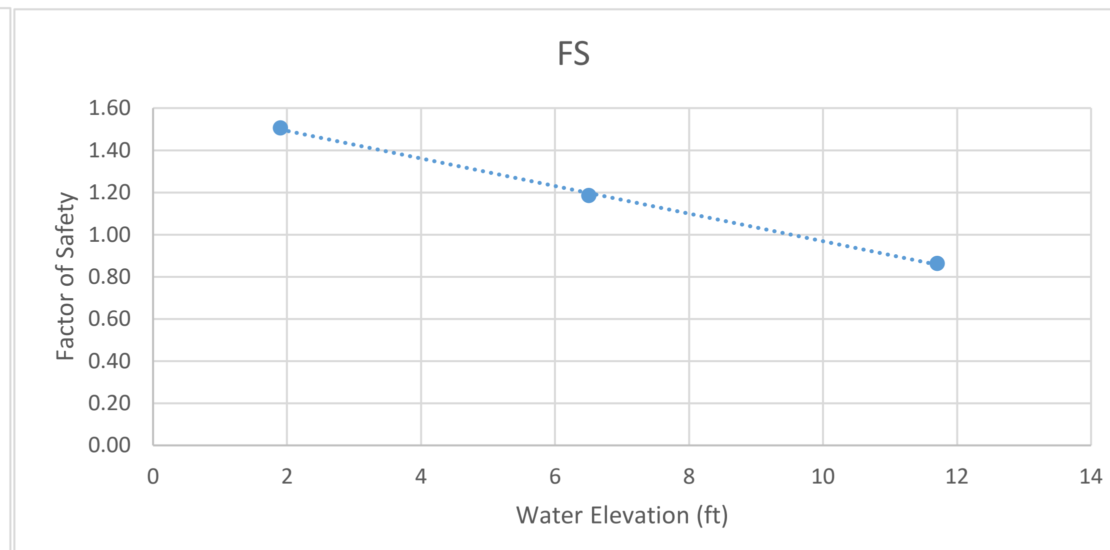
Seepage

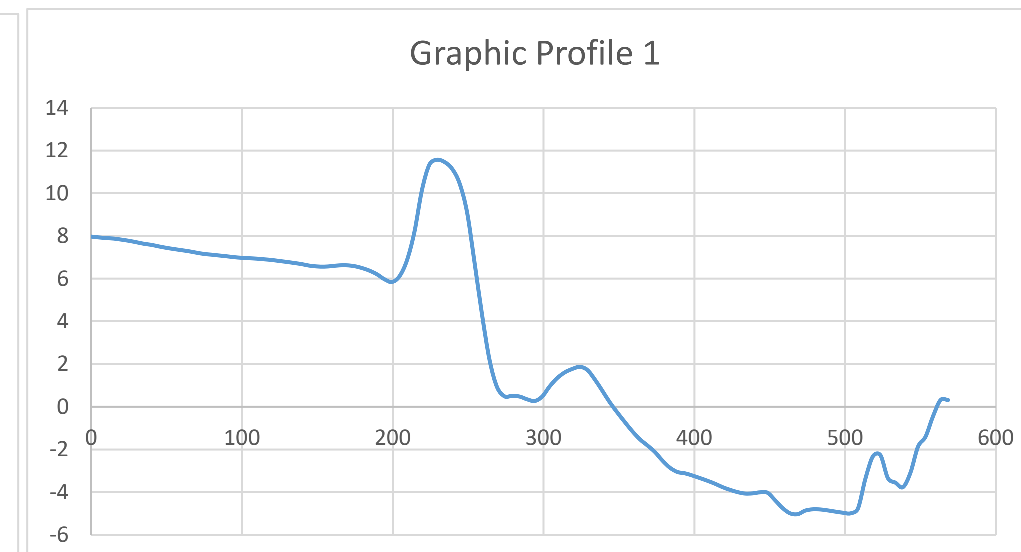
Water Elevation	i
1.9	0.02
6.5	0.12
11.7	0.15

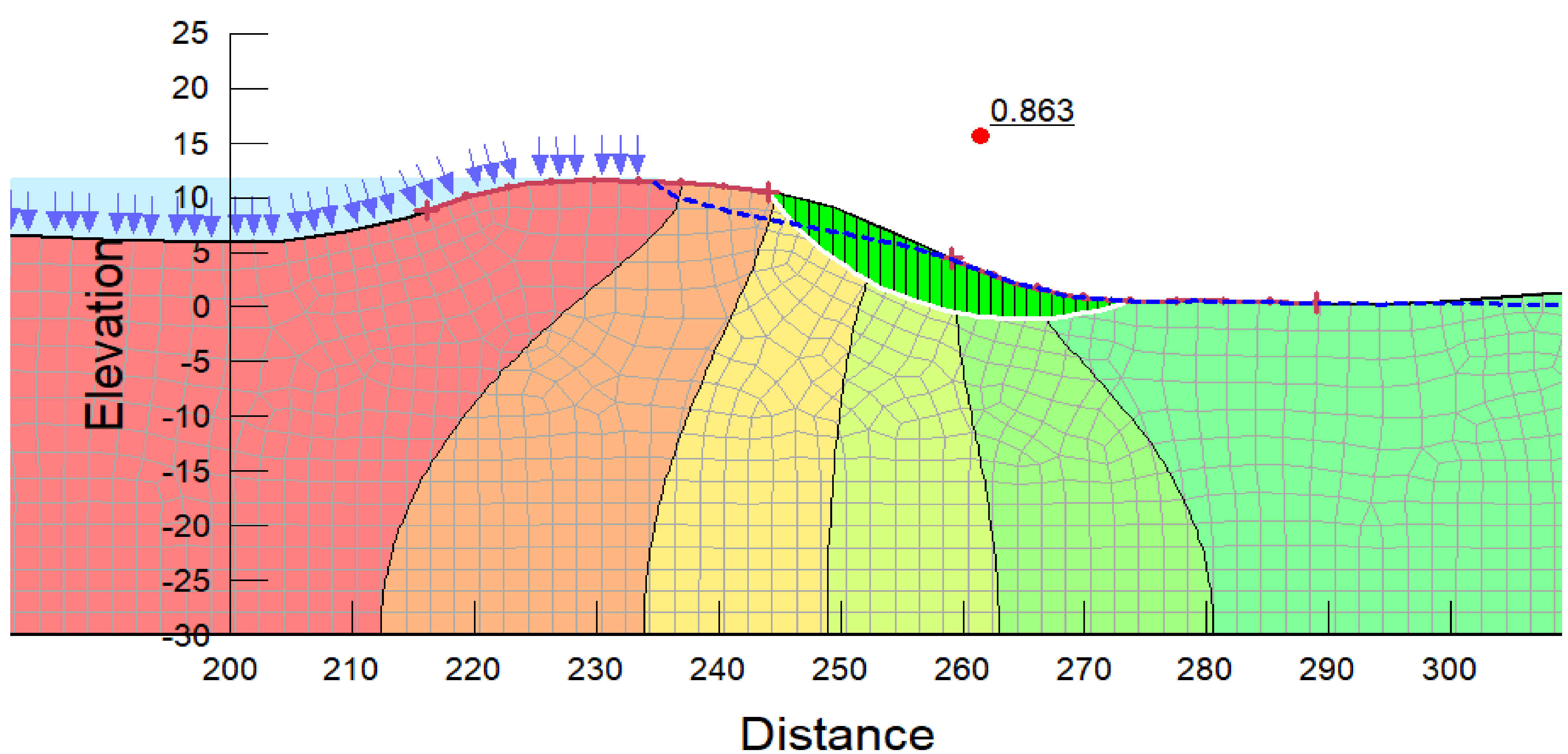
Slope Stability

Water Elevation	FS
1.9	1.51
6.5	1.19
11.7	0.86









Calculated by: SP 6/28/2019
Checked by: JWS 9/10/2019

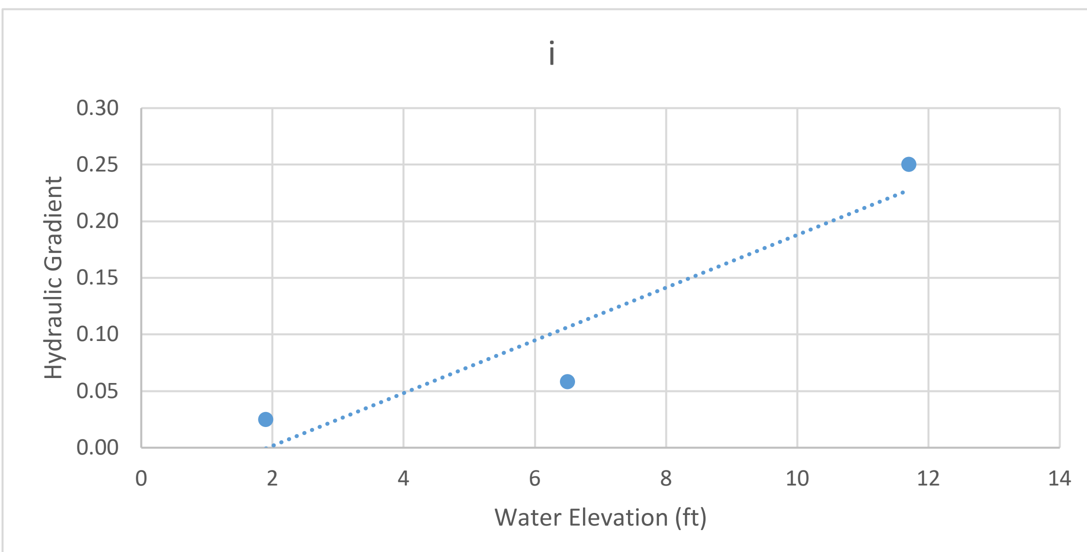
Water El	X	Y	H	dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2	dH	dL	i
	321.5505	4.9475667	0.755742	4.96748	4.337145	24.67586	18.81083	43.48669	6.594444	0.16393547	6.594444	0.02486
1.9	316.583	0.61042129	0.919677									
	323.2079	4.5207	4.5207	6.62496	3.910279	43.8901	15.29028	59.18037	7.692878	0.4487408	7.692878	0.058332
6.5	316.583	0.61042129	4.969441									
	321.5505	4.9475667	4.947567	4.96748	4.337145	24.67586	18.81083	43.48669	6.594444	1.6499267	6.594444	0.2502
11.7	316.583	0.61042129	6.597493									

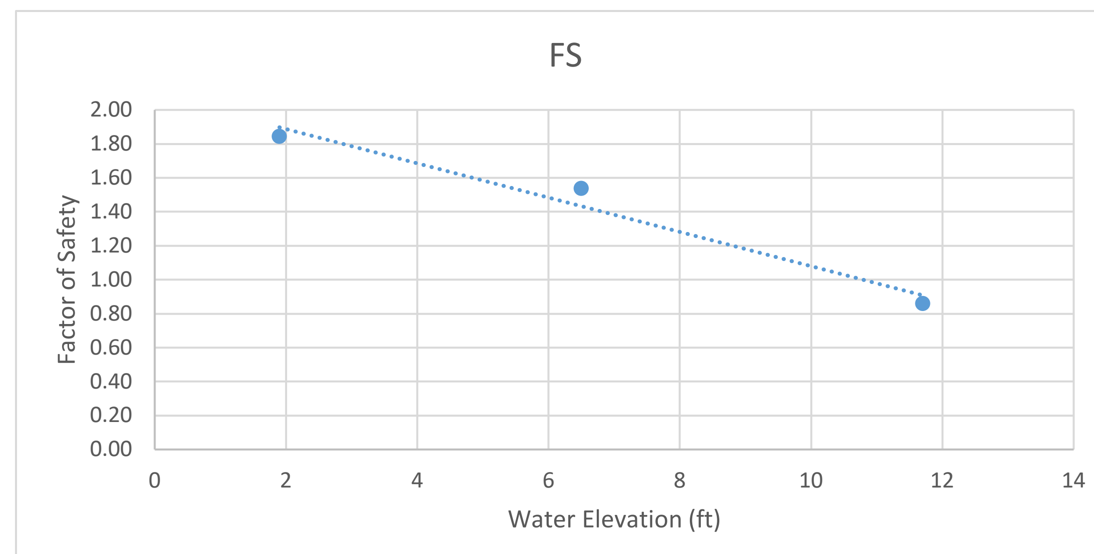
Seepage

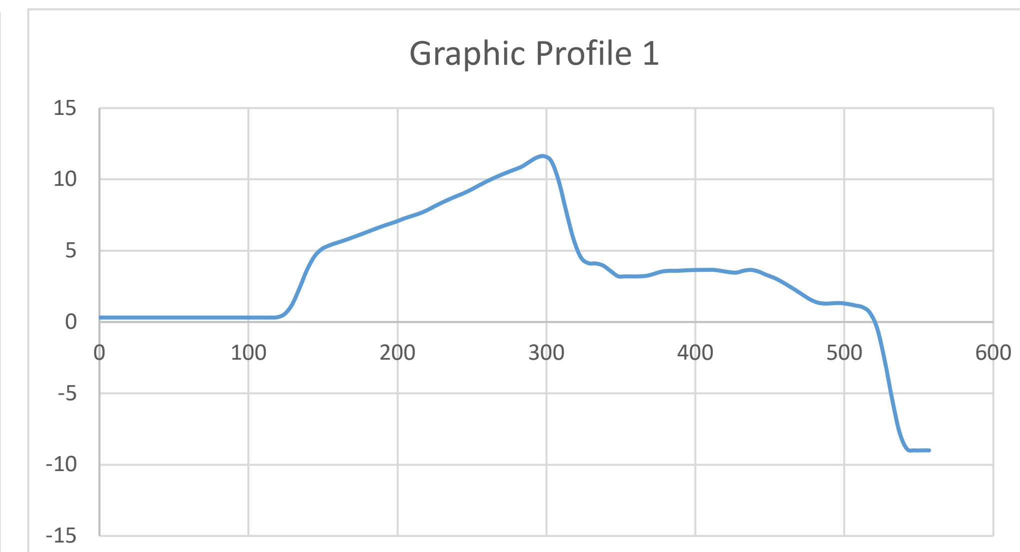
Water Elevation	i
1.9	0.02
6.5	0.06
11.7	0.25

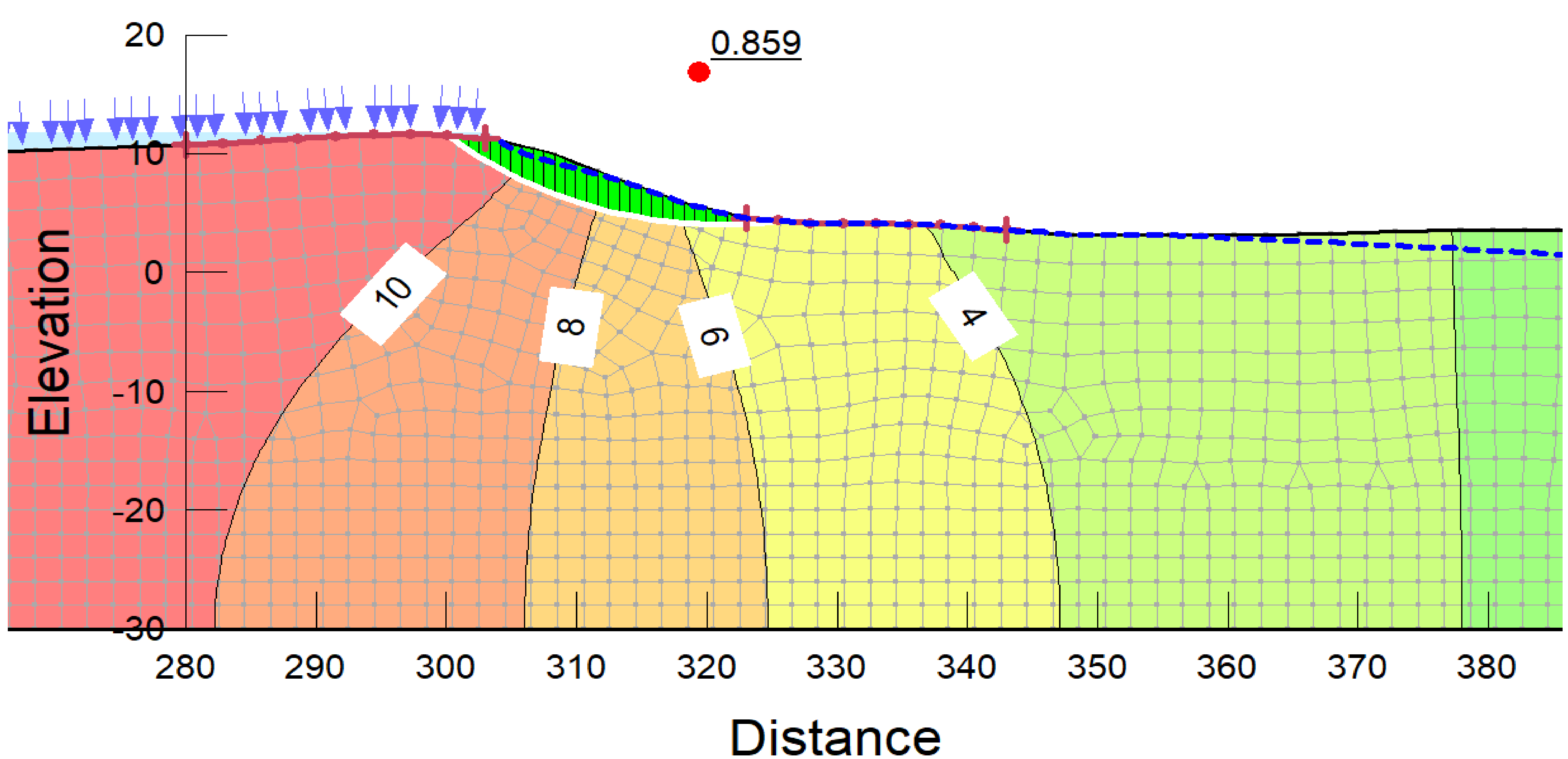
Slope Stability

Water Elevation	FS
1.9	1.84
6.5	1.54
11.7	0.86









Calculated by: SP 6/28/2019
Checked by: JWS 9/10/2019

Water El	X	Y	H	dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2	dH	dL	i
	609.398	5.3632667	1.636515	5.28531	4.064766	27.9345	16.52232	44.45683	6.667595	0.0477107	6.667595	0.007156
1.9	604.1127	1.2985006	1.684226									
	609.398	5.3632667	4.613277	5.28531	4.064766	27.9345	16.52232	44.45683	6.667595	0.2446755	6.667595	0.036696
6.5	604.1127	1.2985006	4.857953									
	609.398	5.3632667	5.363267	5.28531	4.064766	27.9345	16.52232	44.45683	6.667595	1.2988947	6.667595	0.194807
11.7	604.1127	1.2985006	6.662161									

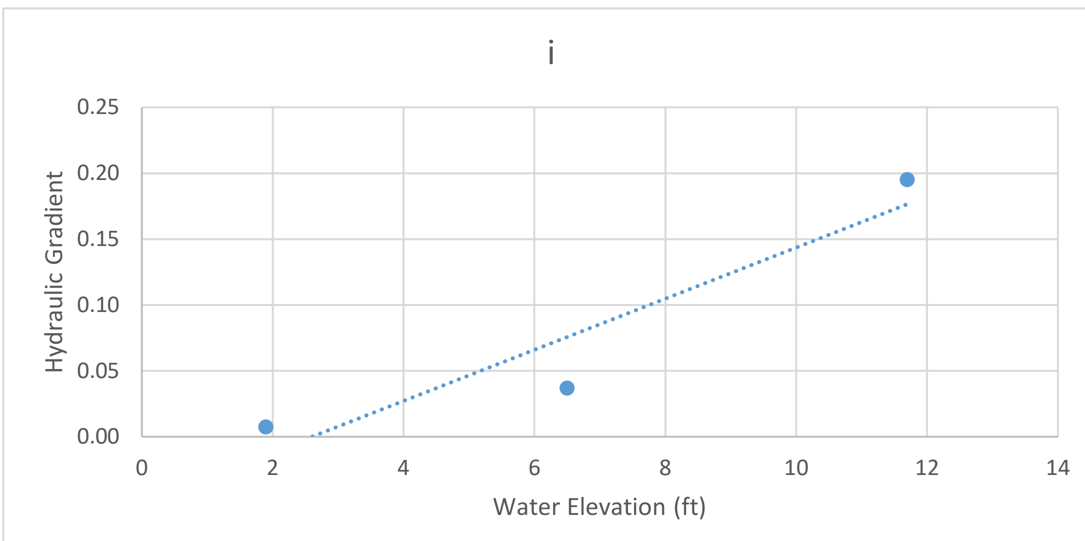
Calculated by: SP 6/28/2019
Checked by: JWS 9/10/2019

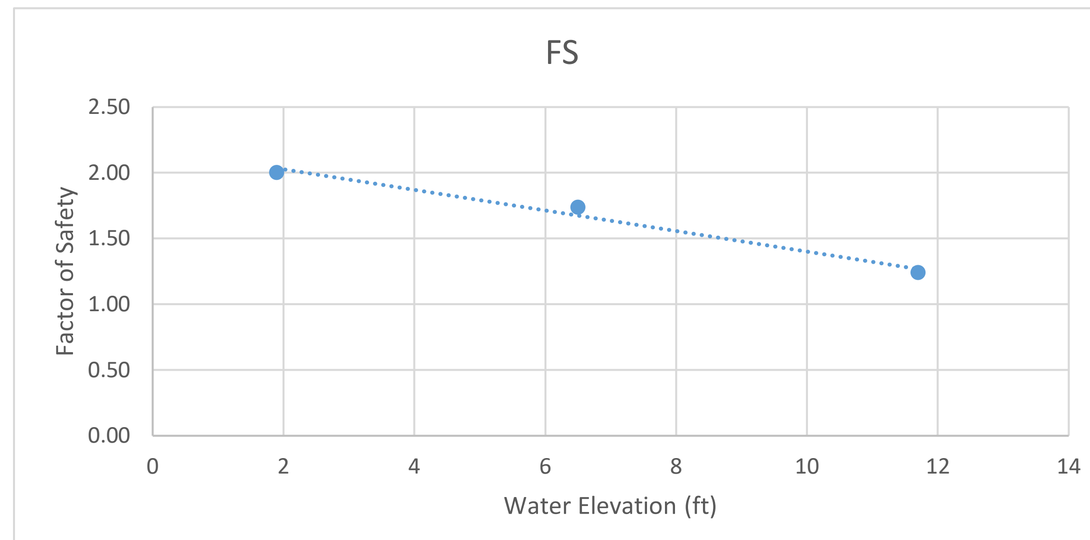
Seepage

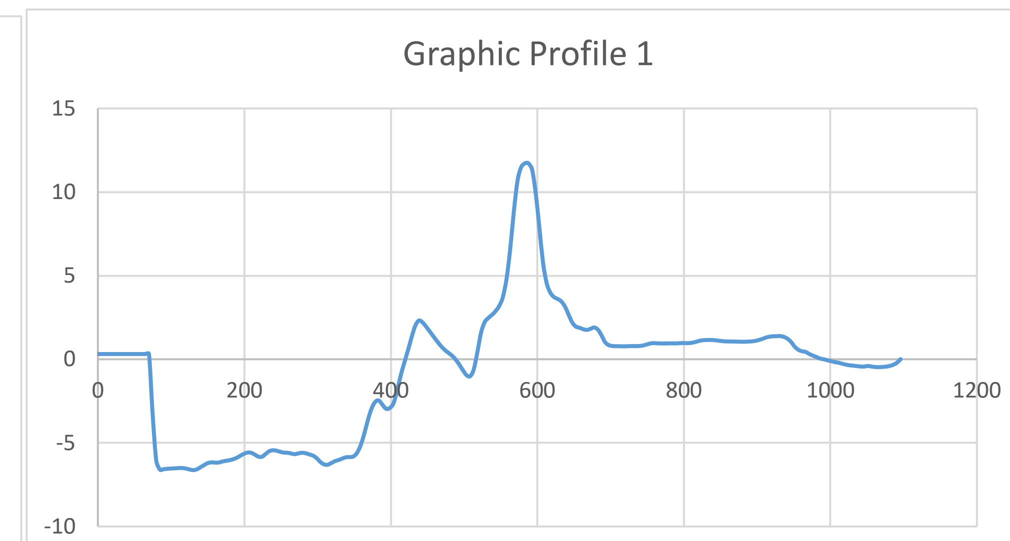
Water Elevation	i
1.9	0.01
6.5	0.04
11.7	0.19

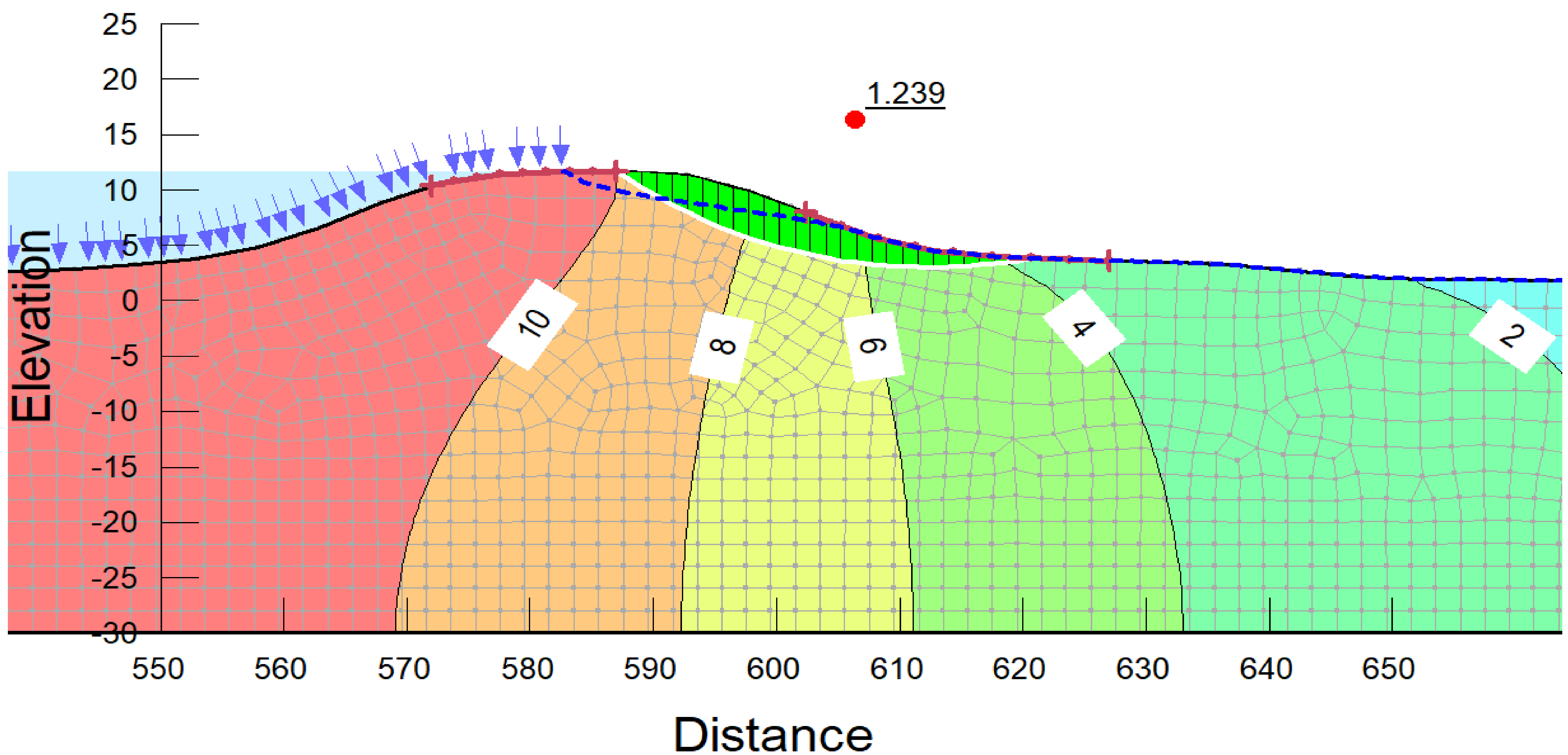
Slope Stability

Water Elevation	FS
1.9	2.00
6.5	1.74
11.7	1.24









Calculated by: SP 6/28/2019
Checked by: JWS 9/10/2019

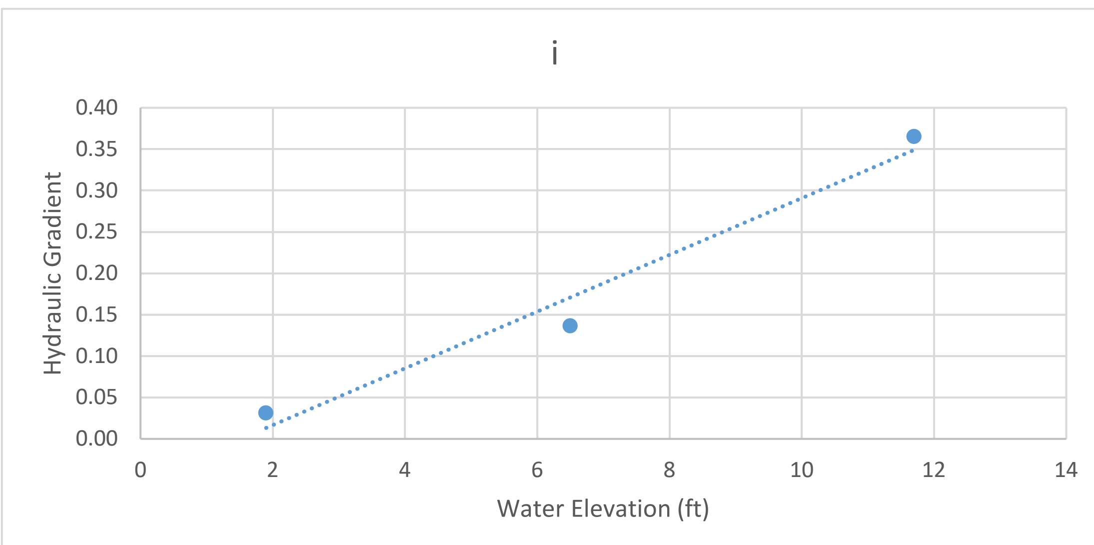
Water El	X	Y	H	dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2	dH	dL	i
	182.8473	2.7901	0.88659924	5.32734	1.22828	28.38055	1.508672	29.88922	5.467104	0.171496	5.467103734	0.031369
1.9	177.52	1.56182	1.0580957									
	182.8473	2.7901	2.2973161	5.32734	1.22828	28.38055	1.508672	29.88922	5.467104	0.74693	5.467103734	0.136623
6.5	177.52	1.56182	3.0442465									
	182.8473	2.7901	2.7901	5.32734	1.22828	28.38055	1.508672	29.88922	5.467104	1.996304	5.467103734	0.365148
11.7	177.52	1.56182	4.7864042									

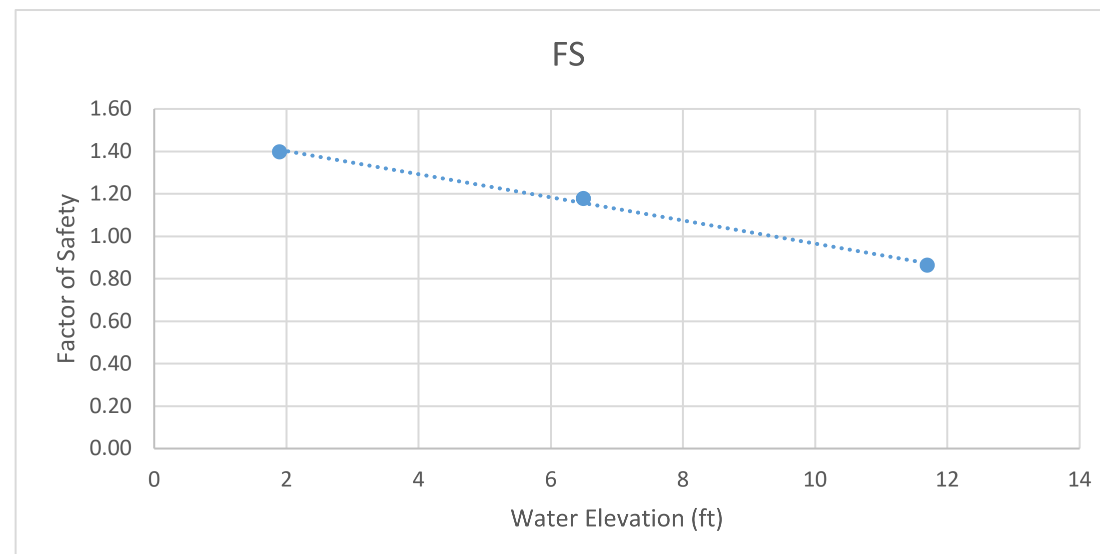
Seepage

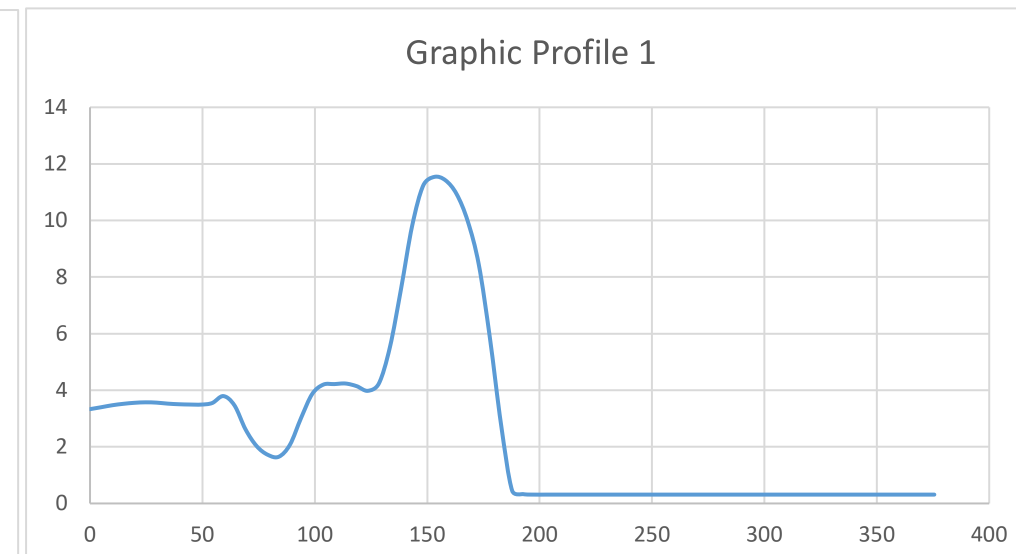
Water Elevation	i
1.9	0.03
6.5	0.14
11.7	0.37

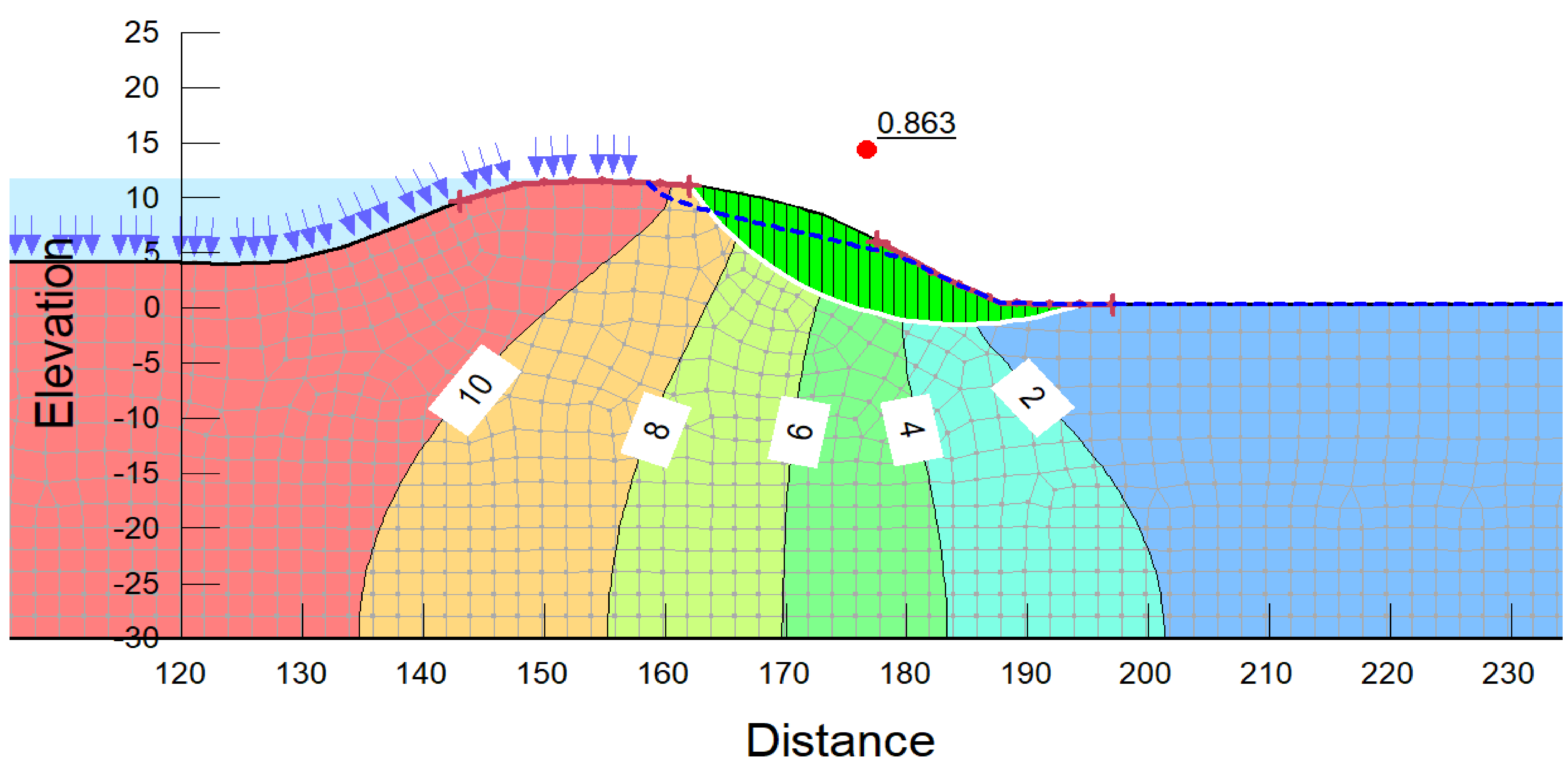
Slope Stability

Water Elevation	FS
1.9	1.40
6.5	1.18
11.7	0.86









Calculated by: SP 6/28/2019
Checked by: JWS 9/10/2019

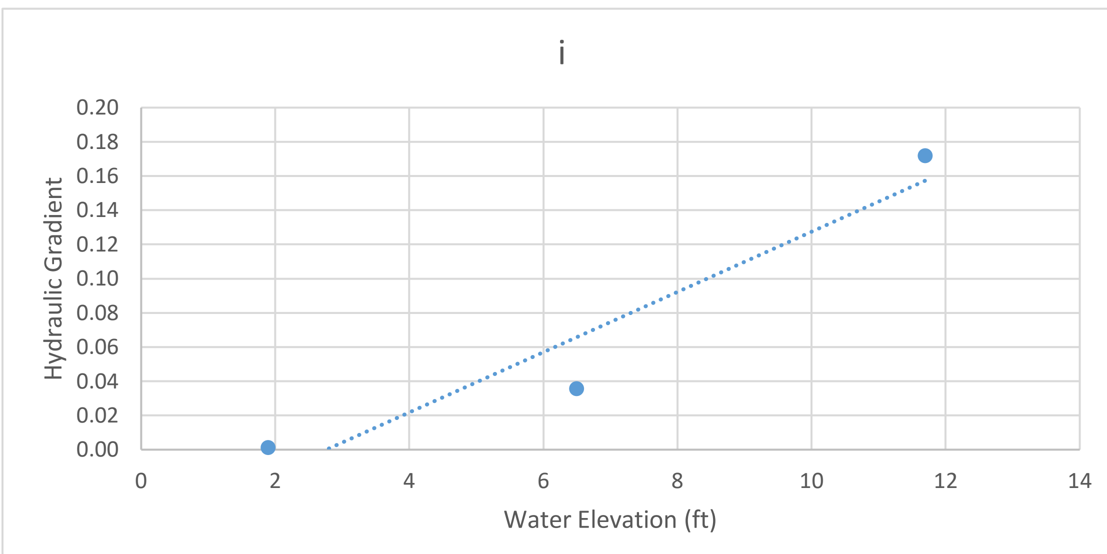
Water El	X	Y	H	dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2	dH	dL	i
	192.2369	4.596	1.8221483	4.96342	6.22899	24.63554	38.80031	63.43585	7.964663	0.00797	7.96466255	0.001001
1.9	187.2735	-1.63299	1.8301184									
	192.2369	4.596	4.2644943	4.96342	6.22899	24.63554	38.80031	63.43585	7.964663	0.28127	7.96466255	0.035315
6.5	187.2735	-1.63299	4.5457646									
	188.9225	5.2732	5.2732	5.33485	5.925477	28.46062	35.11127	63.5719	7.973199	1.36896	7.973198675	0.171695
11.7	183.5877	-0.65228	6.64216									

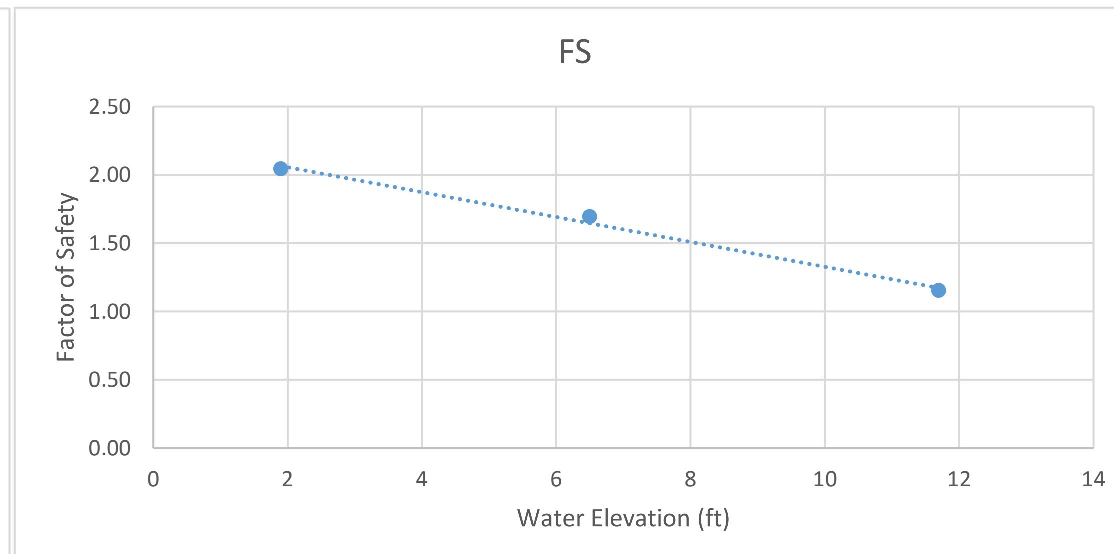
Seepage

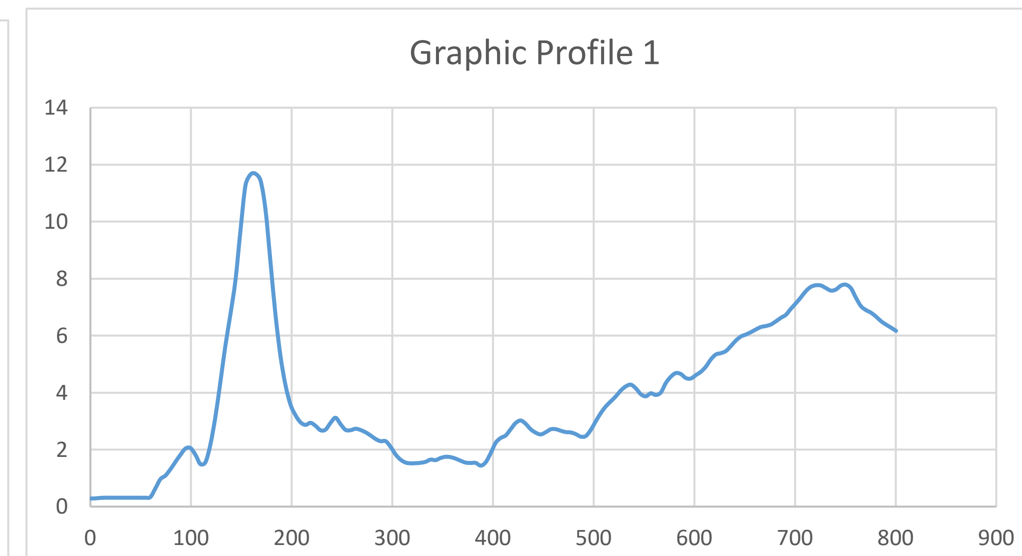
Water Elevation	i
1.9	0.00
6.5	0.04
11.7	0.17

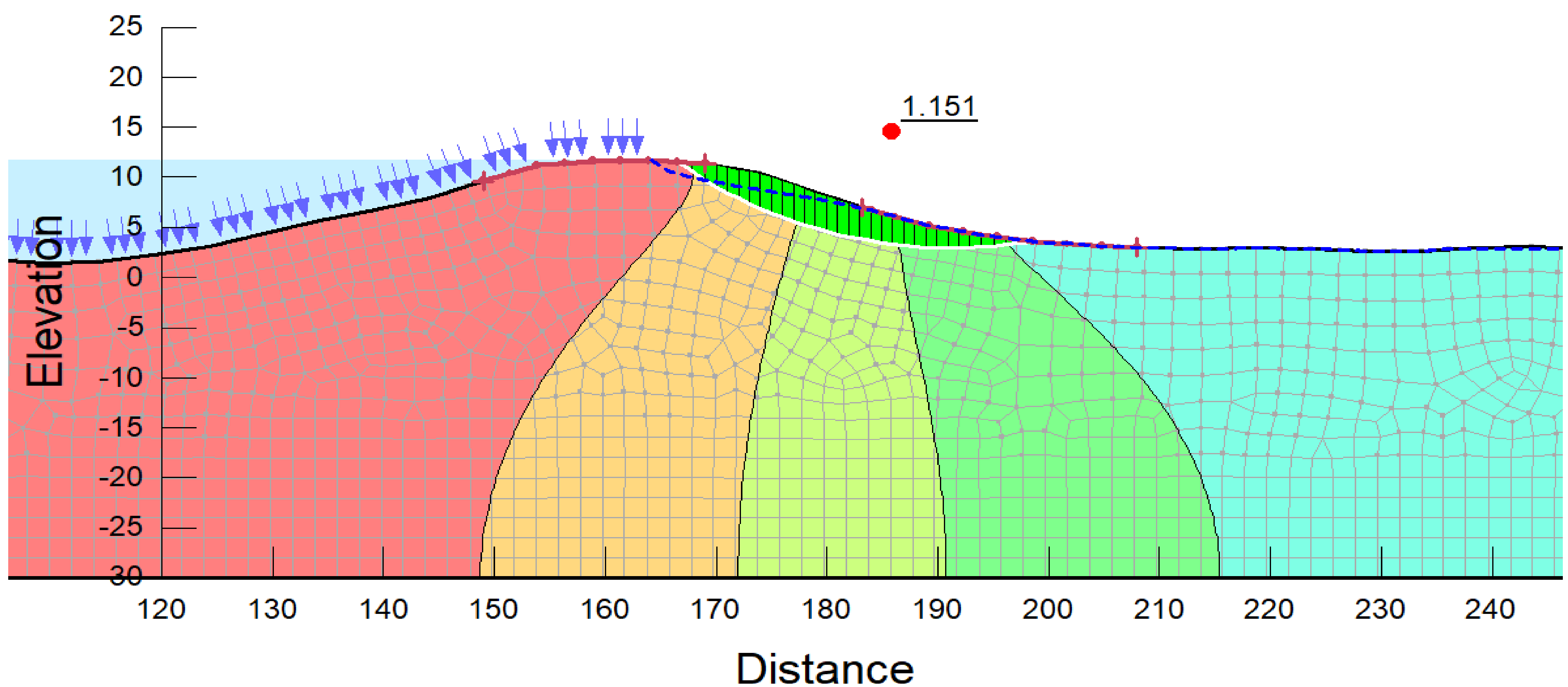
Slope Stability

Water Elevation	FS
1.9	2.04
6.5	1.69
11.7	1.15









Calculated by: SP 6/28/2019
Checked by: JWS 9/10/2019

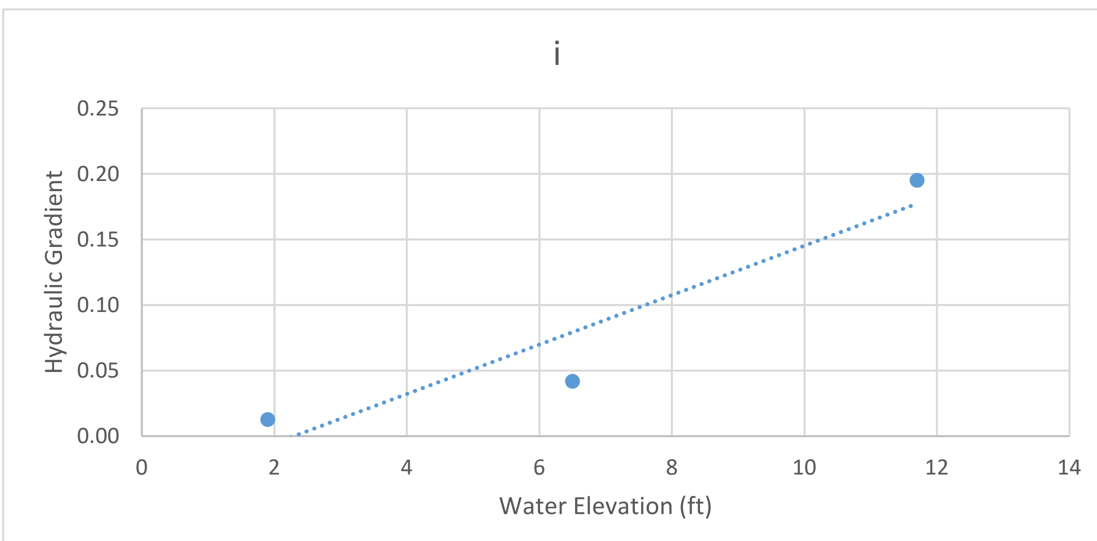
Water El	X	Y	H	dx	dy	dx^2	dy^2	dx^2+dy^2	SQRT(dx^2	dH	dL	i
	513.1856	5.3833333	1.253821	5.16411	4.284695	26.66803	18.35861	45.02665	6.71019	0.0843141	6.71019	0.012565
1.9	508.0215	1.0986381	1.338135									
	513.1856	5.3833333	4.362225	5.16411	4.284695	26.66803	18.35861	45.02665	6.71019	0.2791652	6.71019	0.041603
6.5	508.0215	1.0986381	4.64139									
	513.1856	5.3833333	5.383333	5.16411	4.284695	26.66803	18.35861	45.02665	6.71019	1.3080404	6.71019	0.194933
11.7	508.0215	1.0986381	6.691374									

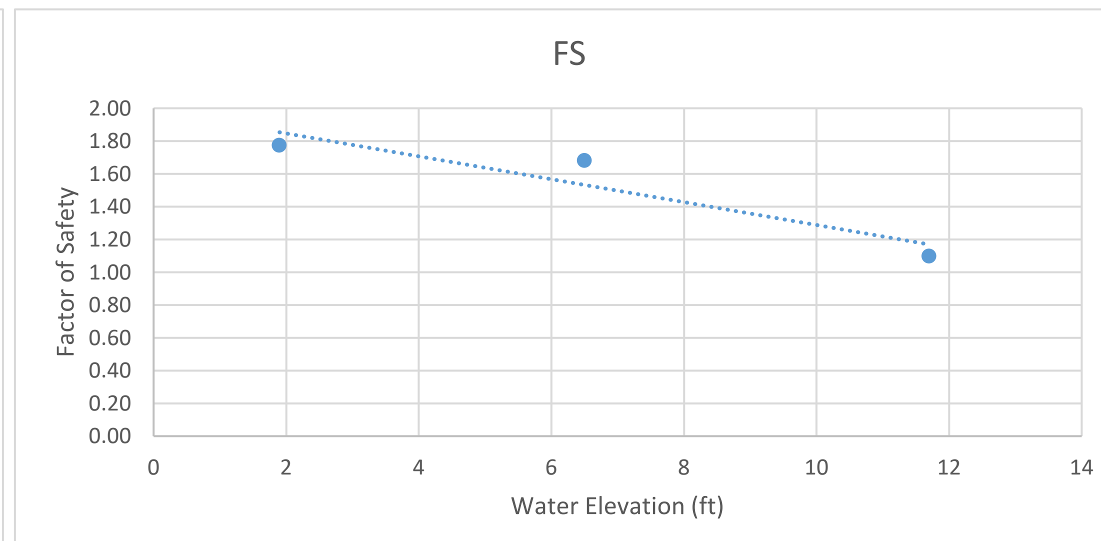
Seepage

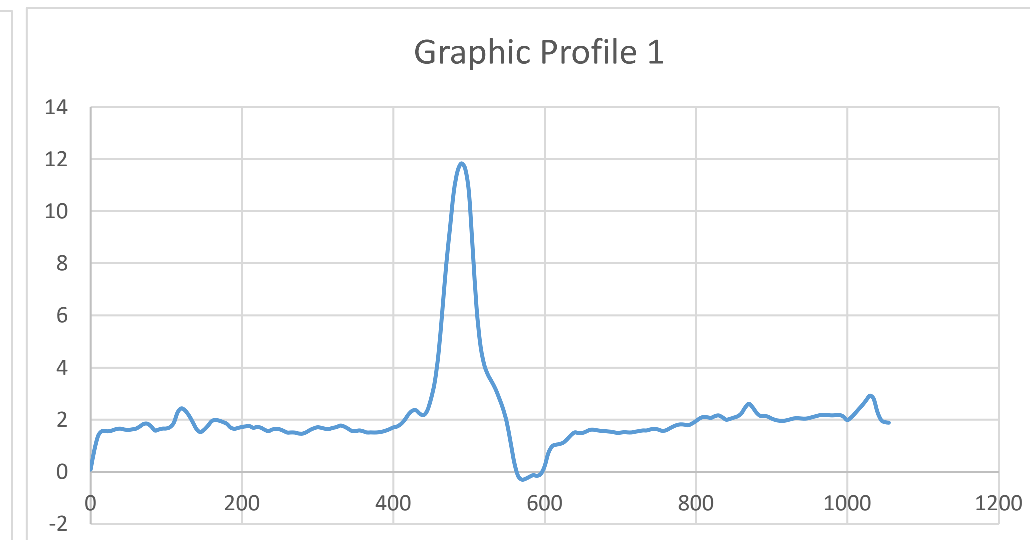
Water Elevation	i
1.9	**0.01**
6.5	**0.04**
11.7	**0.19**

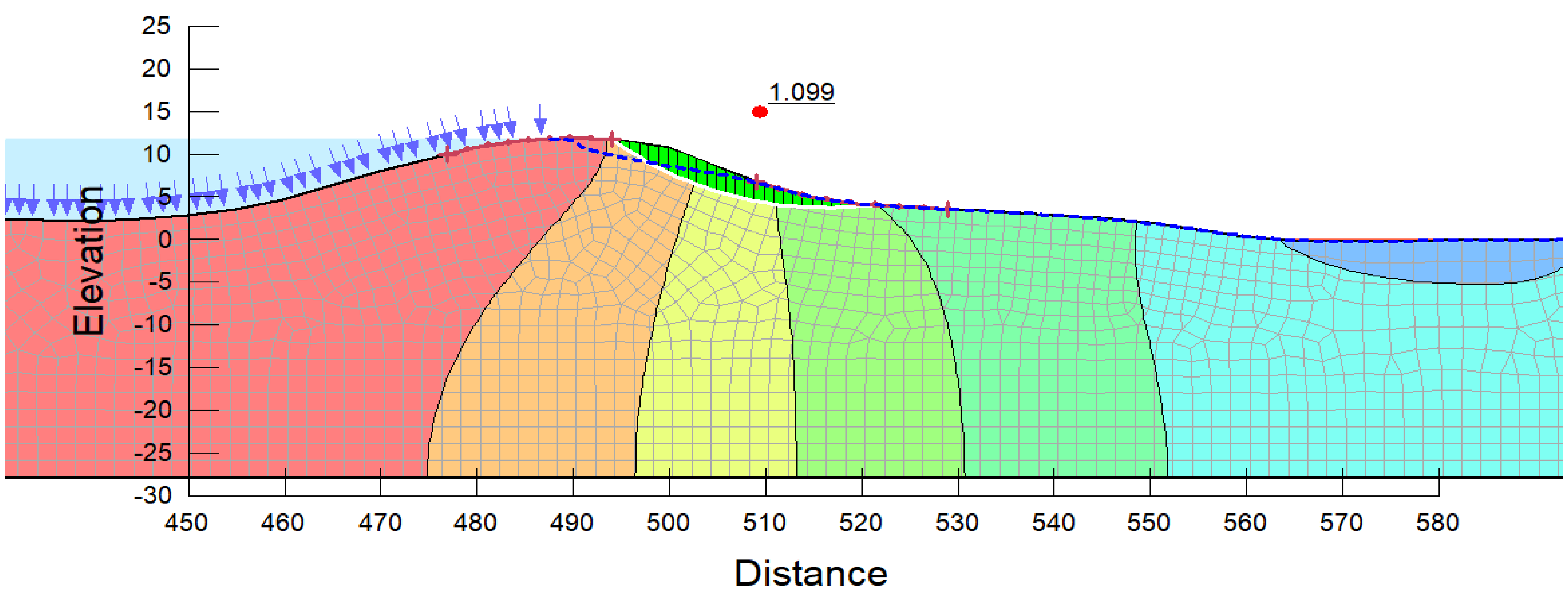
Slope Stability

Water Elevation	FS
1.9	**1.77**
6.5	**1.68**
11.7	**1.10**









Appendix G-7 Earthquake Analysis

Summary

Design accelerations were downloaded from the ASCE 7 Hazard Tool for the sites chosen by Tetra Tech (Attached). However, the design acceleration for ASCE 7 has a return period of approximately 1,500, which may or may not be appropriate for the level of consequences associated the levees in for the Westminster project.

The analysis method for slope stability proposed by Scott Shewbridge (CEIWR) is to calculate a yield acceleration using the following steps:

- Evaluate the Factor of Safety for a number of different levels of pseudo-static acceleration.
- When the FOS=1, that is the yield acceleration. Accelerations above that level will cause yielding and deformation.
- Compare the yield acceleration for the levees with the full range of peak ground accelerations (PGAs), using the full PSHA model from the USGS.
- Use general charts based on the Newmark method to evaluate expected displacements (e.g, Shewbridge et al. 2009, Bray and Travaseroo, etc).

Dr. Shewbridges indicates that if the ky/PGA is greater than 0.5, there is low likelihood of damage. The presence of soft clays with high liquidity indices, or liquefiable silts/sands may mean that the soils have potential for instability. The attached method (Shewbridge et al., 2009) provides an overview of a screening tool developed for levees I California and covers the basics of the analysis.

Dr. Shewbridge indicates that the key to most screening level assessments is not the PGA, but rather the likelihood of soil softening, which is captured in the assessment of ky.

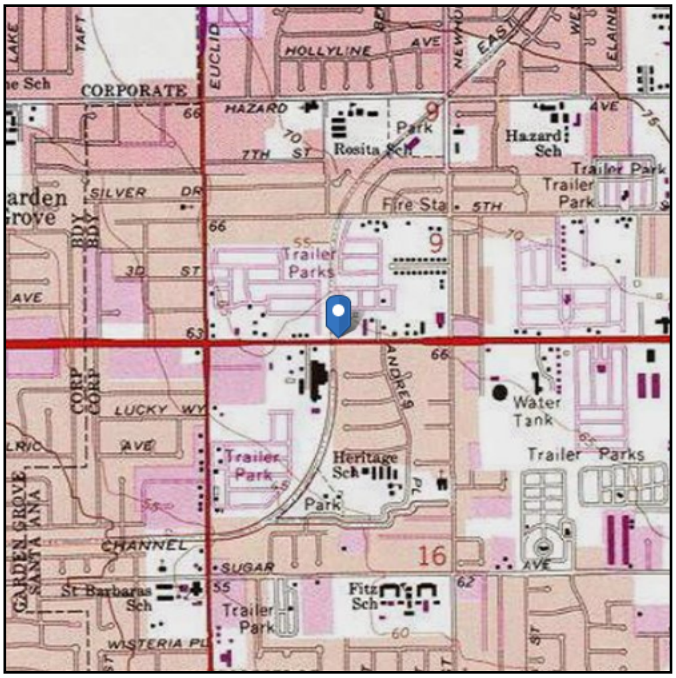
ASCE 7 Hazard Reports

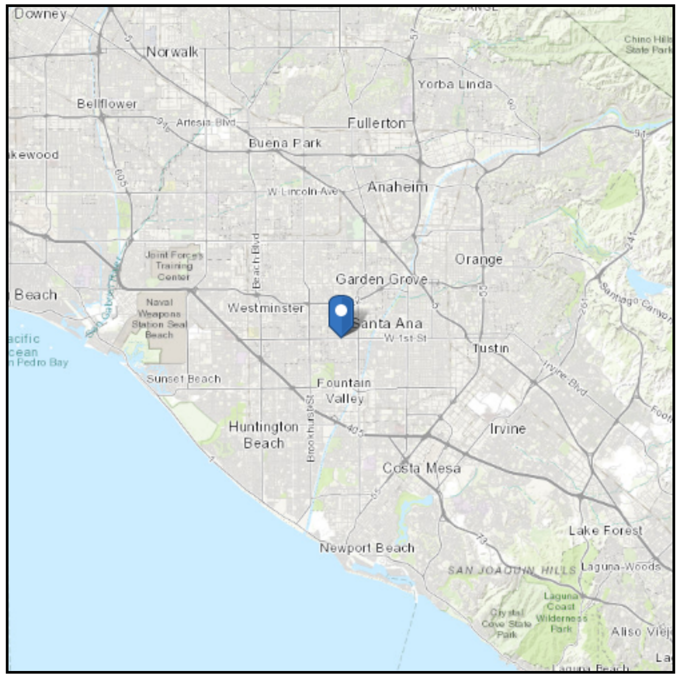
ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: III
Soil Class: D - Default (see Section 11.4.3)

Elevation: 65.03 ft (NAVD 88)
Latitude: 33.745108
Longitude: -117.932655





Wind

Results:

Wind Speed:	101 Vmph
10-year MRI	66 Vmph
25-year MRI	72 Vmph
50-year MRI	76 Vmph
100-year MRI	81 Vmph

Data Source: ASCE/SEI 7-16, Fig. 26.5-1C and Figs. CC.2-1–CC.2-4

Date Accessed: Thu Aug 01 2019

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (annual exceedance probability = 0.000588, MRI = 1,700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

Mountainous terrain, gorges, ocean promontories, and special wind regions should be examined for unusual wind conditions.

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Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	1.33	S_{D1} :	N/A
S_1 :	0.474	T_L :	8
F_a :	1.2	PGA :	0.568
F_v :	N/A	PGA_M :	0.682
S_{MS} :	1.596	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1.25
S_{DS} :	1.064	C_v :	1.366

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

Data Accessed: Thu Aug 01 2019

Date Source: USGS Seismic Design Maps

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Rain

Results:

15-minute Precipitation Intensity: 2.78 in./h

60-minute Precipitation Intensity: 1.34 in./h

Data Source: NOAA National Weather Service, Precipitation Frequency Data Server, Atlas 14 (https://www.nws.noaa.gov/oh/hdsc/)

Date Accessed: Thu Aug 01 2019

Flood

Results:

Flood Zone Categorization: A

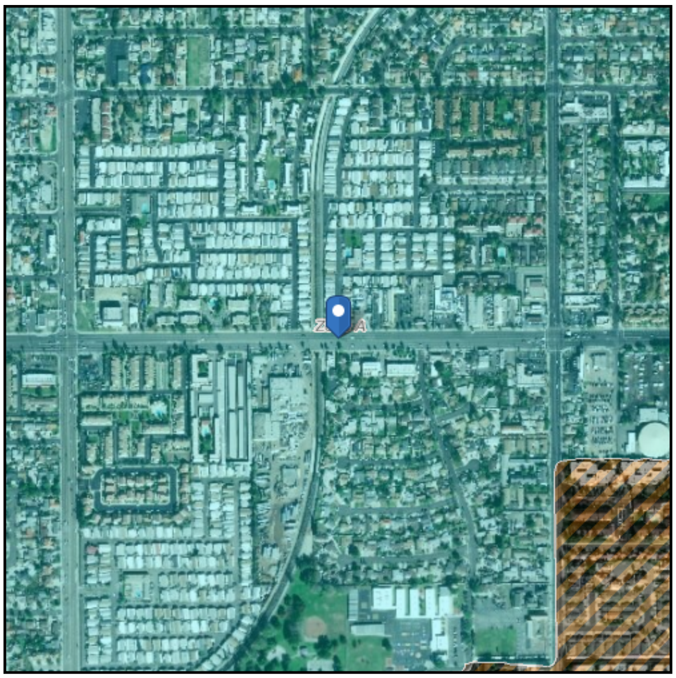
Base Flood Elevation: Refer to map for local elevations and interpolate according to the Authority Having Jurisdiction.

Data Source: FEMA National Flood Hazard Layer - Effective Flood Hazard Layer for US, where modernized (https://msc.fema.gov/portal/search)

Date Accessed: Thu Aug 01 2019

FIRM Panel: If available, download FIRM panel here

Insurance Study Note: Download FEMA Flood Insurance Study for this area here



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

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

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

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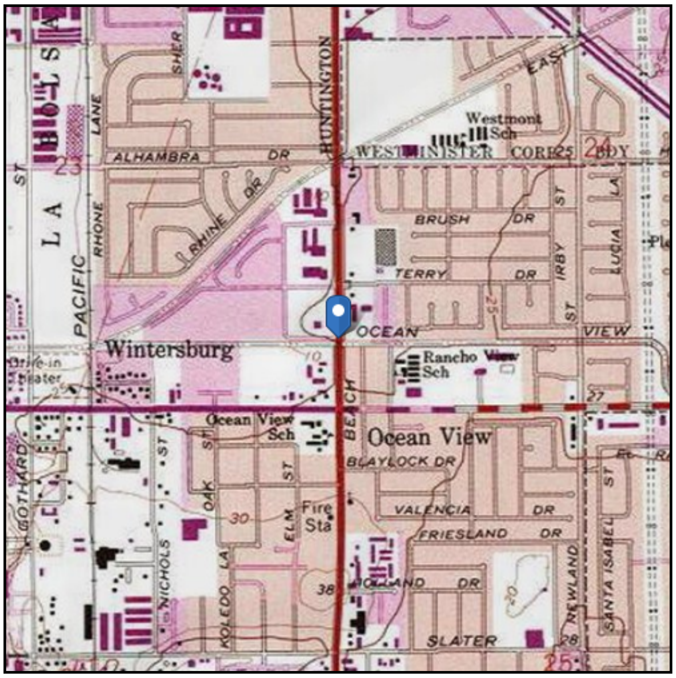
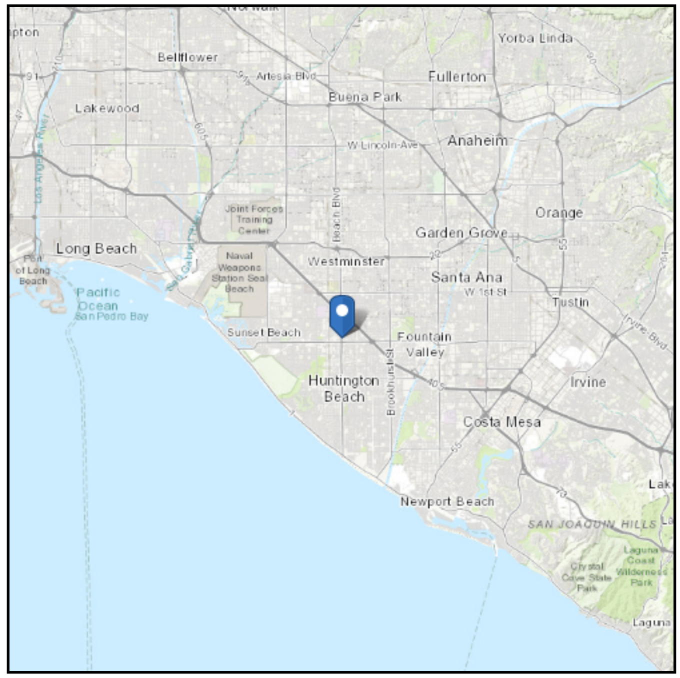
ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: III
Soil Class: D - Default (see Section 11.4.3)

Elevation: 27.54 ft (NAVD 88)
Latitude: 33.717663
Longitude: -117.98909





Wind

Results:

Wind Speed:	101 Vmph
10-year MRI	66 Vmph
25-year MRI	72 Vmph
50-year MRI	76 Vmph
100-year MRI	81 Vmph
Data Source:	ASCE/SEI 7-16, Fig. 26.5-1C and Figs. CC.2-1–CC.2-4
Date Accessed:	Thu Aug 01 2019

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (annual exceedance probability = 0.000588, MRI = 1,700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

Mountainous terrain, gorges, ocean promontories, and special wind regions should be examined for unusual wind conditions.

Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	1.401	S_{D1} :	N/A
S_1 :	0.505	T_L :	8
F_a :	1.2	PGA :	0.605
F_v :	N/A	PGA_M :	0.726
S_{MS} :	1.681	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1.25
S_{DS} :	1.121	C_v :	1.38

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

Data Accessed: Thu Aug 01 2019

Date Source: USGS Seismic Design Maps

ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Rain

Results:

15-minute Precipitation Intensity: 2.94 in./h

60-minute Precipitation Intensity: 1.41 in./h

Data Source: NOAA National Weather Service, Precipitation Frequency Data Server, Atlas 14 (https://www.nws.noaa.gov/oh/hdsc/)

Date Accessed: Thu Aug 01 2019

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Flood

Results:

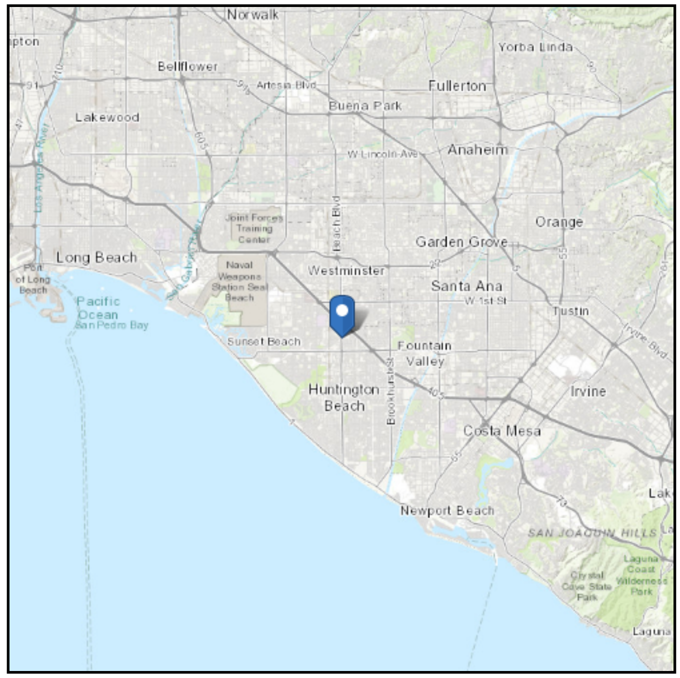
Flood Zone Categorization:	A
Base Flood Elevation:	Refer to map for local elevations and interpolate according to the Authority Having Jurisdiction.
Data Source:	FEMA National Flood Hazard Layer - Effective Flood Hazard Layer for US, where modernized (https://msc.fema.gov/portal/search)
Date Accessed:	Thu Aug 01 2019
FIRM Panel:	If available, download FIRM panel here
Insurance Study Note:	Download FEMA Flood Insurance Study for this area here

ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: III
Soil Class: D - Default (see Section 11.4.3)

Elevation: 26.82 ft (NAVD 88)
Latitude: 33.722996
Longitude: -117.989151



Wind

Results:

Wind Speed:	101 Vmph
10-year MRI	66 Vmph
25-year MRI	72 Vmph
50-year MRI	76 Vmph
100-year MRI	81 Vmph
Data Source:	ASCE/SEI 7-16, Fig. 26.5-1C and Figs. CC.2-1–CC.2-4
Date Accessed:	Thu Aug 01 2019

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (annual exceedance probability = 0.000588, MRI = 1,700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

Mountainous terrain, gorges, ocean promontories, and special wind regions should be examined for unusual wind conditions.

Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	1.399	S_{D1} :	N/A
S_1 :	0.504	T_L :	8
F_a :	1.2	PGA :	0.604
F_v :	N/A	PGA_M :	0.724
S_{MS} :	1.678	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1.25
S_{DS} :	1.119	C_v :	1.38

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

Data Accessed: Thu Aug 01 2019

Date Source: USGS Seismic Design Maps

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Rain

Results:

15-minute Precipitation Intensity: 2.94 in./h

60-minute Precipitation Intensity: 1.41 in./h

Data Source: NOAA National Weather Service, Precipitation Frequency Data Server, Atlas 14 (https://www.nws.noaa.gov/oh/hdsc/)

Date Accessed: Thu Aug 01 2019

Flood

Results:

Flood Zone Categorization: A

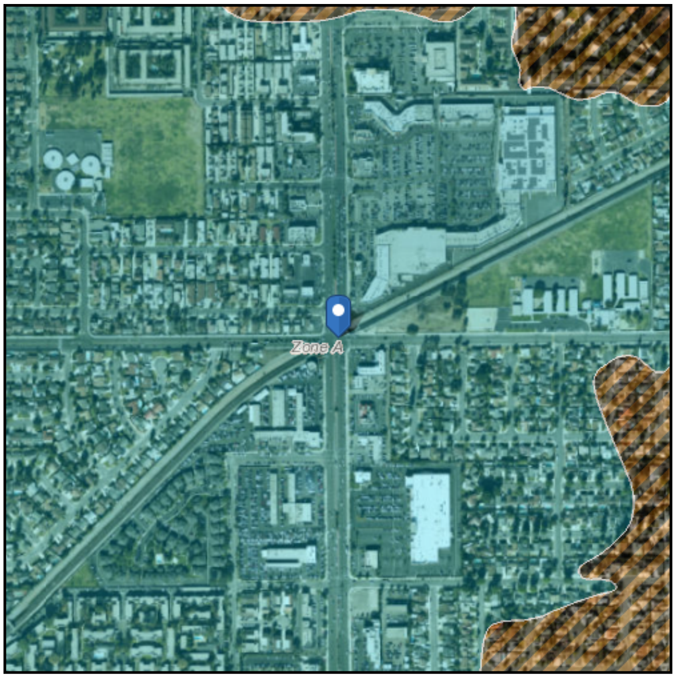
Base Flood Elevation: Refer to map for local elevations and interpolate according to the Authority Having Jurisdiction.

Data Source: FEMA National Flood Hazard Layer - Effective Flood Hazard Layer for US, where modernized (https://msc.fema.gov/portal/search)

Date Accessed: Thu Aug 01 2019

FIRM Panel: If available, download FIRM panel here

Insurance Study Note: Download FEMA Flood Insurance Study for this area here



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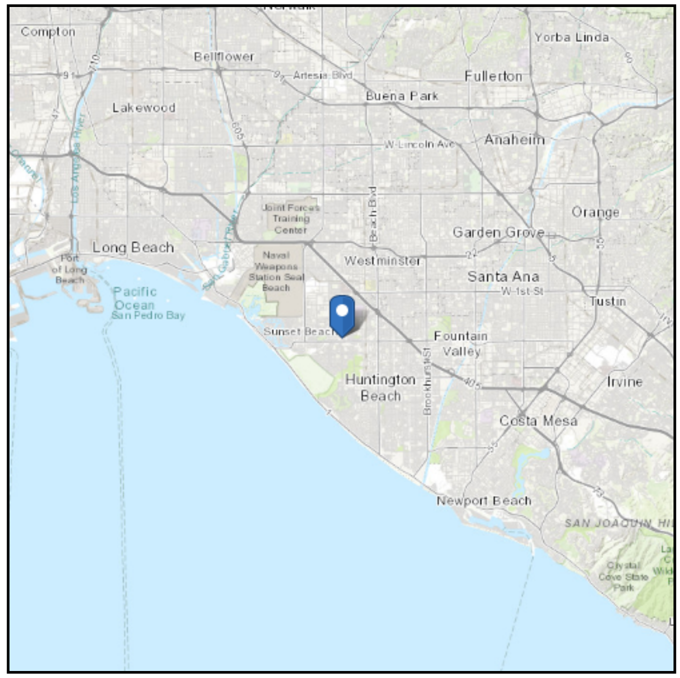
ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: III
Soil Class: D - Default (see Section 11.4.3)

Elevation: 12.47 ft (NAVD 88)
Latitude: 33.717193
Longitude: -118.015241



Wind

Results:

Wind Speed:	101 Vmph
10-year MRI	66 Vmph
25-year MRI	72 Vmph
50-year MRI	76 Vmph
100-year MRI	81 Vmph
Data Source:	ASCE/SEI 7-16, Fig. 26.5-1C and Figs. CC.2-1–CC.2-4
Date Accessed:	Thu Aug 01 2019

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (annual exceedance probability = 0.000588, MRI = 1,700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

Mountainous terrain, gorges, ocean promontories, and special wind regions should be examined for unusual wind conditions.

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Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	1.431	S_{D1} :	N/A
S_1 :	0.518	T_L :	8
F_a :	1.2	PGA :	0.62
F_v :	N/A	PGA_M :	0.744
S_{MS} :	1.717	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1.25
S_{DS} :	1.145	C_v :	1.386

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

Data Accessed: Thu Aug 01 2019

Date Source: USGS Seismic Design Maps

Rain

Results:

15-minute Precipitation Intensity: 2.98 in./h

60-minute Precipitation Intensity: 1.43 in./h

Data Source: NOAA National Weather Service, Precipitation Frequency Data Server, Atlas 14 (https://www.nws.noaa.gov/oh/hdsc/)

Date Accessed: Thu Aug 01 2019

Flood

Results:

Flood Zone Categorization: A

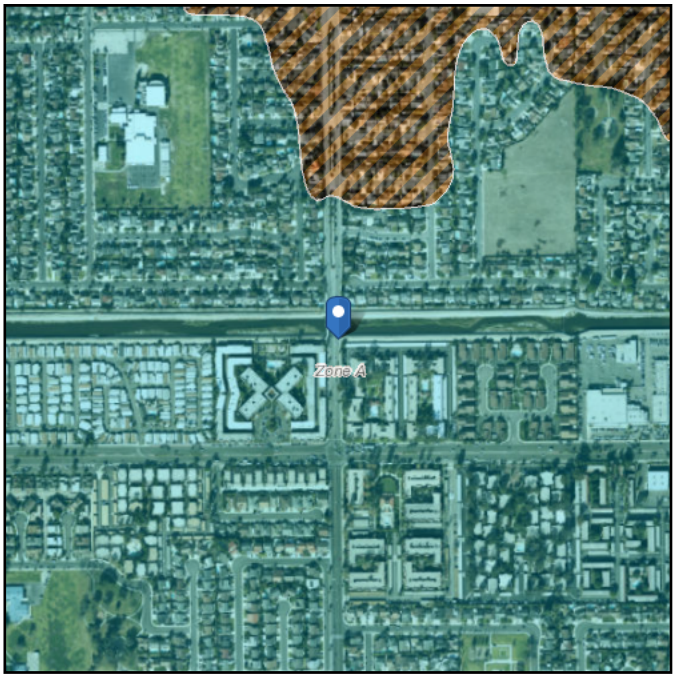
Base Flood Elevation: Refer to map for local elevations and interpolate according to the Authority Having Jurisdiction.

Data Source: FEMA National Flood Hazard Layer - Effective Flood Hazard Layer for US, where modernized (https://msc.fema.gov/portal/search)

Date Accessed: Thu Aug 01 2019

FIRM Panel: If available, download FIRM panel here

Insurance Study Note: Download FEMA Flood Insurance Study for this area here

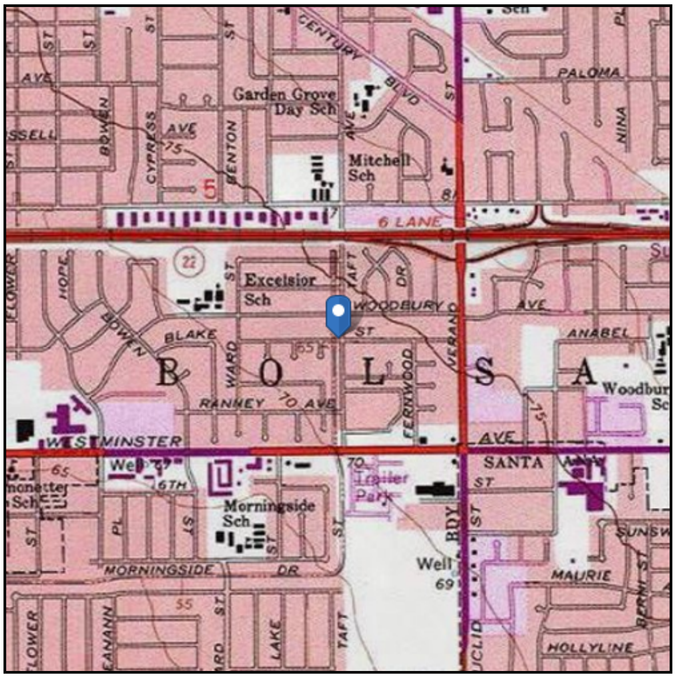


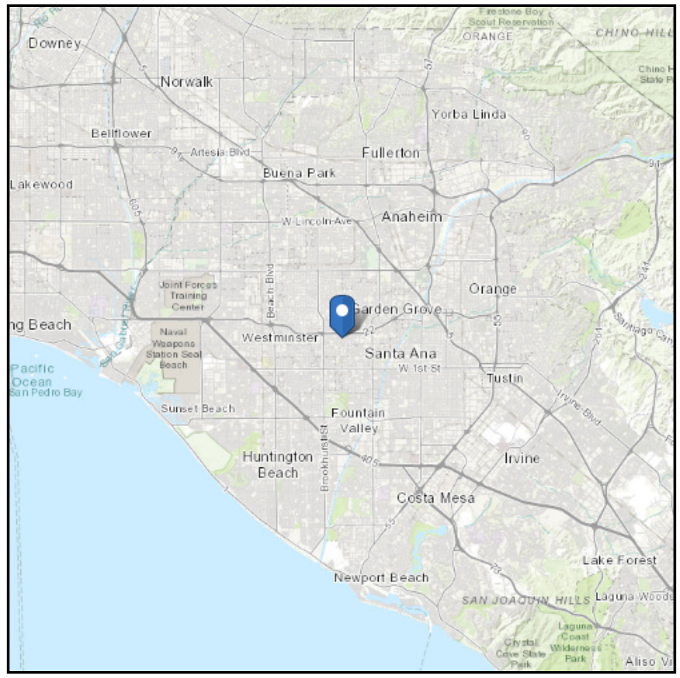
ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: III
Soil Class: D - Default (see Section 11.4.3)

Elevation: 74.3 ft (NAVD 88)
Latitude: 33.762795
Longitude: -117.942016





Wind

Results:

Wind Speed:	101 Vmph
10-year MRI	66 Vmph
25-year MRI	72 Vmph
50-year MRI	76 Vmph
100-year MRI	81 Vmph
Data Source:	ASCE/SEI 7-16, Fig. 26.5-1C and Figs. CC.2-1–CC.2-4
Date Accessed:	Thu Aug 01 2019

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (annual exceedance probability = 0.000588, MRI = 1,700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

Mountainous terrain, gorges, ocean promontories, and special wind regions should be examined for unusual wind conditions.

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Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	1.355	S_{D1} :	N/A
S_1 :	0.481	T_L :	8
F_a :	1.2	PGA :	0.578
F_v :	N/A	PGA_M :	0.694
S_{MS} :	1.626	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1.25
S_{DS} :	1.084	C_v :	1.371

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

Data Accessed: Thu Aug 01 2019

Date Source: USGS Seismic Design Maps

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Rain

Results:

15-minute Precipitation Intensity: 2.78 in./h

60-minute Precipitation Intensity: 1.34 in./h

Data Source: NOAA National Weather Service, Precipitation Frequency Data Server, Atlas 14 (https://www.nws.noaa.gov/oh/hdsc/)

Date Accessed: Thu Aug 01 2019

ASCE
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Flood

Results:

Flood Zone Categorization: X (shaded)

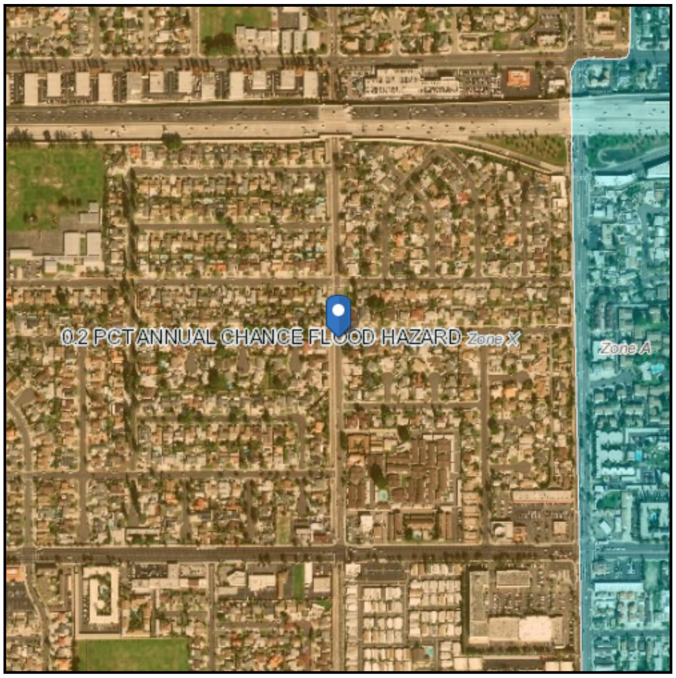
Base Flood Elevation: Refer to map for local elevations and interpolate according to the Authority Having Jurisdiction.

Data Source: FEMA National Flood Hazard Layer - Effective Flood Hazard Layer for US, where modernized (https://msc.fema.gov/portal/search)

Date Accessed: Thu Aug 01 2019

FIRM Panel: If available, download FIRM panel here

Insurance Study Note: Download FEMA Flood Insurance Study for this area here



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"Simplified approach to assess levee seismic vulnerability," Shewbridge et al., 2009

SIMPLIFIED APPROACH TO ASSESS LEVEE SEISMIC VULNERABILITY

Scott Shewbridge, PhD, PE, GE, Jerry Wu PhD, PE, GE, [1] Sujan Punyamurthula, PhD, PE, GE, Juan Vargas, PE, [2] Steve Mahnke, PE, Mike Inamine, PE [3]

ABSTRACT

This paper presents an overview of the California Department of Water Resources Urban Levee Program simplified seismic vulnerability method, its development, analysis protocols, and typical results. The methodology is based on a Newmark-type of deformation evaluation, with specialized charts to evaluate levee response to seismic loading, similar to the widely-used Makdisi-Seed simplified approach to evaluate dams. Example seismic hazard, cyclic-stress ratio, maximum seismic loading and deformation estimation charts are presented with details on how they are used to evaluate levee seismic vulnerability in a screening level program.

INTRODUCTION

The State of California's Department of Water Resources (DWR) is undertaking unprecedented efforts to evaluate and upgrade aging and deteriorating levees along the Sacramento and San Joaquin River Flood Control Projects in the Central Valley and Delta. Of highest priority, DWR is fully evaluating more than 350 miles of urban "Project" levees in these areas, with plans to survey the entire 1,600 miles of "Project" levees in the Central Valley. One of the factors being evaluated is seismic vulnerability.

To expedite a screening level assessment of potential impacts, a simplified method for evaluating seismic performance has been developed (URS, 2008). The assessment process uses probabilistic hazard and finite element analyses to develop simplified charts for evaluating levels of shaking, liquefaction triggering, post-liquefaction soil strengths, and estimated magnitudes of induced displacement. The vulnerability of the levee is then categorized based on the expected deformation and the ability to provide flood protection after a seismic event.

APPROACH

The methodology is comprised of four sets of analyses:

- Seismic loading – evaluated using *newly developed* seismic hazard maps and cyclic stress ratio and maximum seismic coefficient charts.

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[2] URS Corporation, 2870 Gateway Oaks Drive, Suite 150, Sacramento, CA, sujan_punyamurthula@urscorp.com and juan_vargas@urscorp.com
[3] Department of Water Resources, Division of Flood Management, 2825 Watt Avenue #100, Sacramento, CA 95821 inamine@water.ca.gov and gmahnke@water.ca.gov

- Liquefaction - evaluated using conventional published liquefaction triggering and post-triggering soil strength correlations.
- Seismic slope stability and yield acceleration – evaluated using conventional slope stability programs.
- Deformation - evaluated using *newly developed* levee deformation charts.

Using the results from the above analyses, the levees are then classified using a four-tiered Seismic Vulnerability Classification System, giving engineers and policy makers information on the expected performance of levees and a means to prioritize areas for future actions.

SEISMIC HAZARD AND GROUND MOTION EVALUATIONS

A probabilistic seismic hazard analysis was performed during the recently-completed DWR Delta Risk Management Study Phase 1 Project to evaluate the potential for seismic hazard in the Sacramento-San Joaquin River Delta (URS 2007a and b). The results from this study were expanded to include all of the areas being considered in the DWR Urban Levee Evaluation Program. An example of one of these Peak Horizontal Acceleration (PHA) hazard maps is presented on Figure 1. Maps such as this are used to estimate the probabilistic peak horizontal acceleration for different return periods for each levee location.

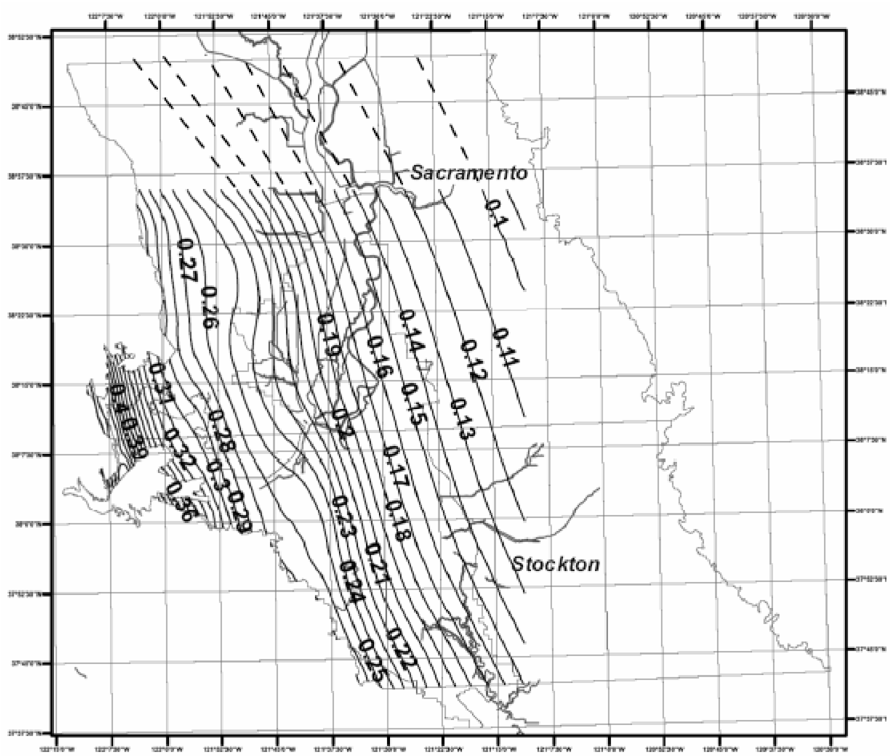

Figure 1 – 200-year Return Period Average Peak Horizontal Acceleration (PHA) Map for the Sacramento and San Joaquin Delta Area

MODELING OF INDUCED SEISMIC LOADS

For the finite element modeling conducted to evaluate seismically induced loads in levees, several recorded strike-slip earthquake motions were selected to represent expected events of different magnitudes and distances throughout the study areas and are summarized in the following table:

Earthquake Motion Characteristic	Superstition Hills 1987	Landers 1992	Denali 2002	Imperial Valley 1979	Denali 2002
Station	USGS 5052 Plaster City	CDMG 12331 Hemet Fire Station	Alyeska TAPS Pump Station 9	USGS 286 Superstition Mountain Camera	Alyeska TAPS Pump Station No. 8
Component (degree)	045/135	000/090	013/103	045/135	049/319
Magnitude (M)	6.5	7.3	7.9	6.5	7.9
Closest distance (km)	22.2	68.7	54.8	24.6	104.9
Mechanism	Strike-slip	Strike-slip	Strike-slip	Strike-slip	Strike-slip
Vs30 (m/sec)	345	339	383	362	425
PHA (g)	0.121/0.186 [1]	0.081/0.097 [1]	0.056/0.075 [1]	0.109/0.195	0.046/0.036
PGV (cm/sec)	9.48/20.64 [1]	5.60/5.65[1]	11.42/12.12[1]	5.14/8.77	5.26/4.42
PGD (cm)	1.91/5.43 [1]	2.07/2.24[1]	8.98/11.04[1]	2.22/2.78	3.61/3.10
Arias Intensity (m/sec)	0.30/0.63 [1]	0.26/0.24[1]	0.21/0.20[1]	0.09/0.20	0.05/0.04

Notes:
V_{s30} Average shear wave velocity in top 30m
PHA Peak horizontal acceleration
PGV Peak ground velocity
PGD Peak ground displacement

[1]Dual values are for the two horizontal components, respectively.

Athanasopolous (2008) has recently run additional analyses using the same levee finite element models (to be described below), evaluating response to over 400 additional earthquake motions. The initial results indicate very good agreement with the results for the limited number of motions used to develop the charts used for this study and presented herein.

DEVELOPMENT OF REPRESENTATIVE REGIONAL LEVEE AND FOUNDATION MODELS

To assess the impact of varying levee and foundation conditions throughout the study area, three levee and foundation models representing conditions in the northern (Marysville), central (West Sacramento), and southern (Stockton) areas of the Central Valley region were developed and used for dynamic response and Newmark-type analyses.

The idealized subsurface profile for the Marysville area model consists of 25 feet high levees, underlain by 15 feet of relatively weak, fine-grained Holocene sediments (which may include relatively recent mining sediments), overlying 30 feet of looser sandy and gravelly soil. Underlying Pleistocene sediments are located about 70 feet beneath the levee crest and are modeled as dense coarse-grained. The West Sacramento area model consists of a 25-foot high silty levee overlying 60-foot-thick Holocene sediments of medium dense sand, followed by Pleistocene sediments of stiff clay. The Stockton area model consists of a 10 feet high levee, underlain by 80 feet of Holocene sediments consisting of soft, young marsh deposits towards the surface and gravelly, older deposits toward the bottom. This is underlain by relatively stiff Pleistocene sediments about 90 feet beneath the levee crest, modeled as mainly stiff clay. In the finite element model, the Pleistocene sediments are founded on a transmitting boundary.

The profile (or geometry) of existing levees varies by levee location. To limit the number of variables during the finite element dynamic analyses, a representative levee section with 25-foot-wide crest and 2H:1V slope was selected on the basis of engineering judgment. This representative levee section was used to develop finite element models as described in the following section. Estimates of seismic and static stresses will be slightly lower for levees with slopes flatter than 2H:1V. The channel water was modeled at 10ft. below the levee crest, with a phreatic line sloping down to the landside toe and continuing horizontal to the edge of the model. Figure 2 presents a summary of the different modeled sections.

To evaluate loads on blocks founded on potential failure surfaces in the levee, for each model, four potential sliding blocks were specified: Landside-shallow, Waterside-shallow, Landside-deep, and Waterside-deep. Dimensions and locations of these circular sliding blocks were selected based on engineering judgment and experience.

Two-dimensional dynamic response analyses were performed to estimate cyclic shear stresses in levee and levee foundation materials, as well as to estimate the seismic coefficient (i.e., the average induced seismic acceleration) of the potential sliding blocks using the software program QUAD4M (Hudson et al, 1994). The version of QUAD4MB, with a 2003 correction to the calculation of average acceleration of sliding block, was used. The dynamic stress-strain behavior of the materials is assumed viscoelastic. The elastic modulus and viscous damping of materials are calculated iteratively until they are compatible with computed shear strains. Analyses were performed using the time histories scaled to PHA levels of 0.05 g, 0.1 g, 0.2 g, 0.3 g, 0.4 g, and 0.5 g and input as outcrop motions at the top of the half space (i.e., the transmitting boundary at the base of the modeled Pleistocene sediments).

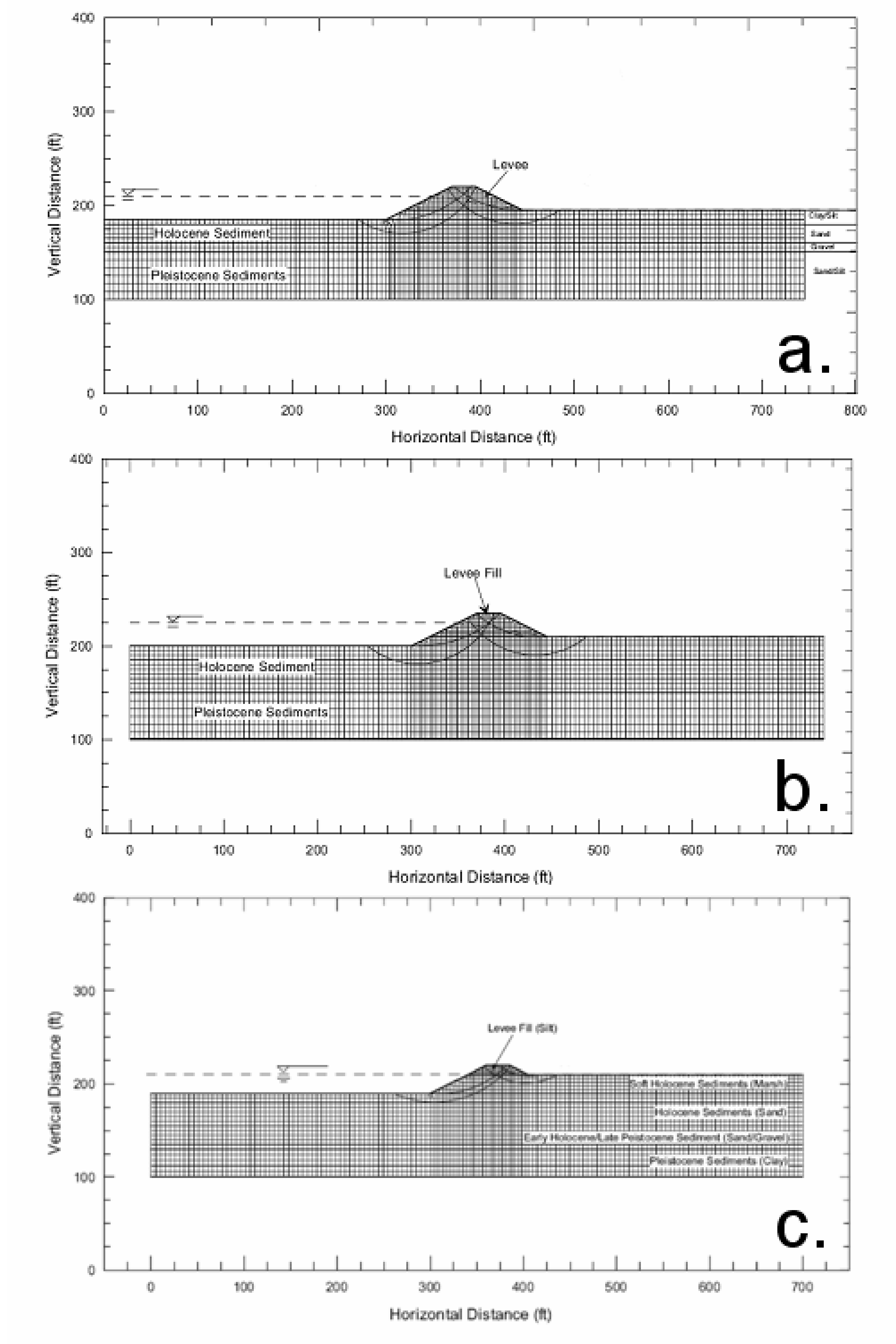

Figure 2 – Levee Finite Element Models for: a) Northern Region (Marysville), b) Central Region (West Sacramento) and c) Southern Region (Stockton).

The material properties used in the QUAD4M analyses include total unit weight (γ), maximum shear modulus (G_{max}), and the modulus reduction (G/G_{max}) and damping ratio (λ) relationships with shear strain. The dynamic parameters were derived as follows:

- Soil unit weights were estimated based on review of laboratory data and engineering judgment.
- Shear wave velocities, Vs were estimated primarily from seismic cone penetrometer tests (CPT) performed during field investigations for the West Sacramento and Stockton areas. Vs for the Marysville area were estimated from standard penetration test (SPT) correlations.
- The lower portion of the Pleistocene sediments was modeled as an elastic half space below the base of the finite element model to account for energy-transmitting characteristics. The half space was assigned the same elastic properties (unit weight and wave velocities) as those of the overlying Pleistocene materials.
- The maximum shear moduli in the materials were obtained based on a compilation of measured values of shear wave velocity and or typical values based on material type and effective overburden stress.
- The variation of shear modulus and damping with shear strain for sandy and gravelly materials is represented by the relationships proposed by Seed and Idriss (1970). The variation of shear modulus and damping for clayey materials is represented by the relationships proposed by Vucetic and Dobry (1991).

For more details on material characterization, see URS 2008.

DEVELOPMENT OF SIMPLIFIED LOAD ESTIMATING CURVES

The results of the dynamic response analyses were used to develop charts to be used in liquefaction triggering analyses, relating cyclic stress ratio (CSR) under the waterside free-field, the crest, and the landside free-field to the PHA. They were also used to develop charts to be used in simplified deformation analyses to evaluate maximum seismic coefficient (K_{max}) for the selected sliding blocks as a function of the PHA.

Simplified Cyclic Stress Ratio (CSR) Curves – The cyclic shear stress ratio is obtained using the following equation from Seed and Idriss (1971):

$$CSR = 0.65\ \tau_{max} / \sigma_v'$$

in which the maximum shear stress, τ_{max}, was calculated in QUAD4M dynamic response analysis, and the effective overburden stress, σ_v', was calculated during a static finite element analysis. Based on compilation and synthesis of results from the various models, average CSR relationships were developed. Figure 3 presents estimates of CSR versus depth for the input accelerations scaled to various values ranging from 0.05 g to 0.5 g at three locations (waterside free field, crest centerline and landside free field) as a function of depth and input motion PHA.

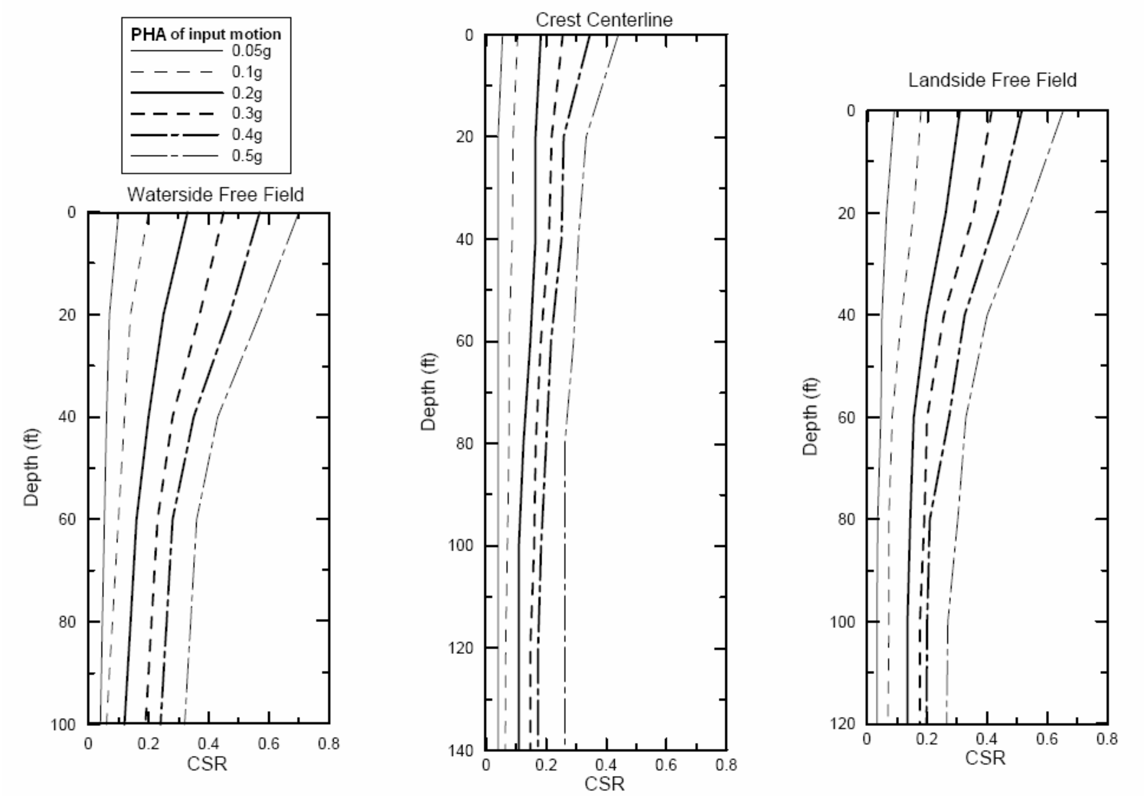

Figure 3 – Cyclic Stress Ratio (CSR) for the Waterside Free Field, Crest Centerline and Landside Free Field as a function of depth and input motion Peak Horizontal Acceleration (PHA).

K_{max} Curves – To assess the seismic loading on blocks sitting on potential failure surfaces within the levee, K_{avg}, the ratio of the horizontal force induced by an earthquake on a block divided by the weight of that block, is calculated from the finite element model results. It is expressed in units of gravity (g) and is a function of time. K_{max} is the maximum value of K_{avg} for each modeled motion and is a measure of the seismic loading for the entire earthquake. Figure 4 presents estimates of K_{max} for blocks founded on different potential failure surfaces (shallow or deep) as a function of depth, apparent foundation stiffness and input motion PHA. Analysts using these curves to estimate seismic loading select the appropriate curve based on a qualitative consideration of the model foundation stiffness. For PHAs less than 0.05g, K_{max} is considered equal to PHA.

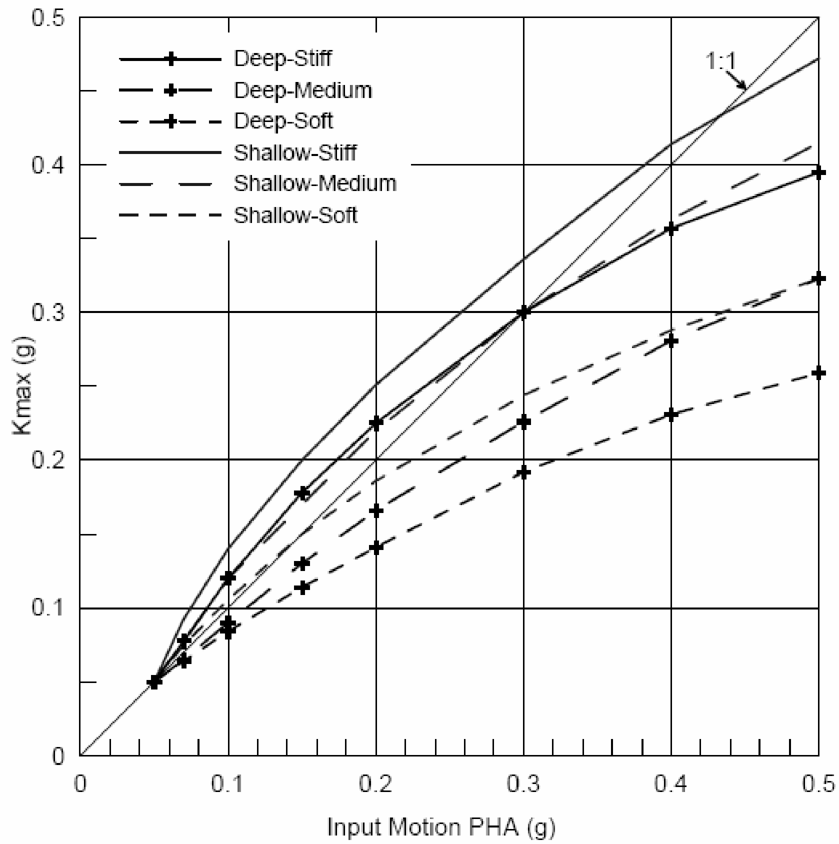

Figure 4 - K_{max} versus Input Motion PHA

While the authors believe that using the above simplified approach to assess K_{max} is appropriate for use in the screening level seismic vulnerability assessment methodology described herein, the influence of differences in waterside and landside behavior and foundation stiffness as affected by foundation sediment stiffness and model asymmetry may both warrant future research and modeling. In fact, levees identified as seismically vulnerable during DWR's Urban levee study are often considered to warrant further evaluation and/or mitigation design using additional, more complex analyses.

NEWMARK DEFORMATION ANALYSIS

Seismically-induced deformations of the levees were evaluated using the sliding block method proposed by Newmark (1965). In the Newmark method, driving inertia forces are represented by the average seismic coefficient (K_{avg}) induced by a design earthquake in the potential slide mass and as calculated by QUAD4M modeling software. Resisting forces are represented by the lowest horizontal acceleration coefficient resulting in deformations to the potential sliding mass. This term is commonly referred to as the yield acceleration, or K_y. K_y can be assessed for any sliding mass in a slope using conventional slope stability methods and is usually found by searching for critical failure surfaces under increasing values of horizontal seismic coefficient until the factor of safety approaches 1.0. Permanent deformations of the potential sliding mass can then be computed by double integrating, with respect to time, the difference between earthquake-induced accelerations (represented by K_{avg}) and K_y. Newmark deformation calculations were performed by using several programs including TNMN, DEFORMP, and a USGS Java-based software program (Jibson and Jibson, 2003).

To develop Newmark displacement relationships for various situations, a suite of K_y values were assumed and used for double integration computations. Assumed K_y values range from 0.05 g to 0.5 g. End users of these Newmark displacement charts will need to calculate site-specific K_y values using appropriate slope stability evaluation methods. Similar to the approach taken by Makdisi and Seed (1978), calculated Newmark displacements are presented in the form of displacement versus K_y / K_{max}. Displacement can then be estimated based on site specific estimated values of K_y, K_{max} and earthquake magnitude using the relationships shown on Figure 5. To simplify the DWR screening process, a single line relationship was selected for each earthquake magnitude, with a bias towards representing the upper range of expected behavior during a Magnitude 7 earthquake. In reality, similar to the results of Makdisi and Seed (1978), the modeled relationships are probably better expressed as bands (URS, 2008).

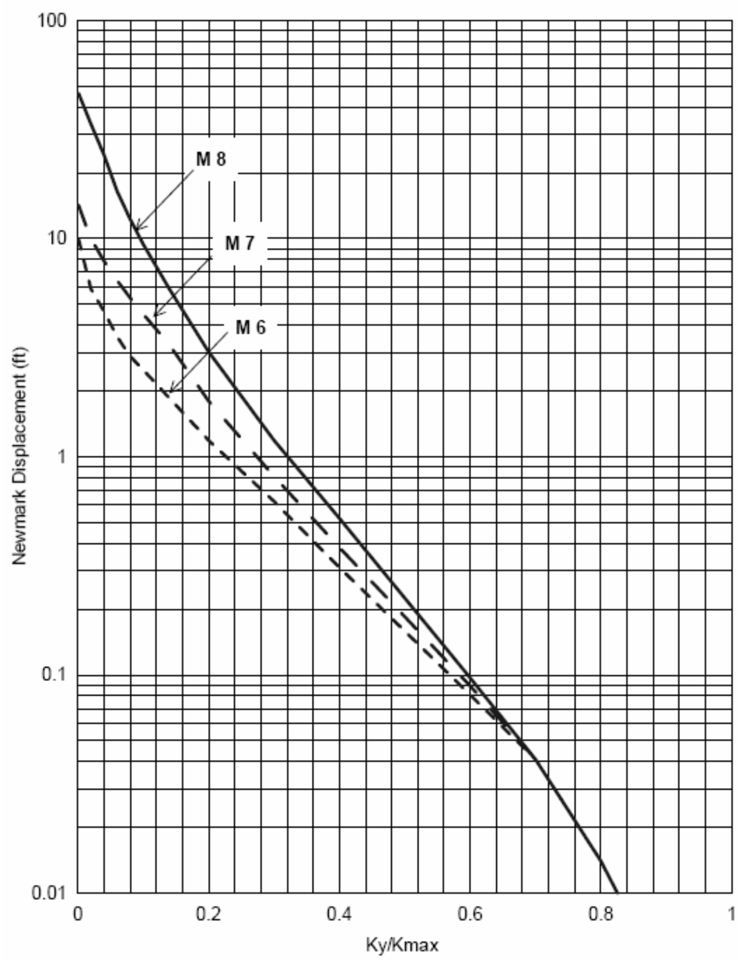

Figure 5 – Newmark Displacement versus K_y /$K_{max.}$

SIMPLIFIED SEISMIC VULNERABILITY ASSESSMENT PROCEDURE

Using the charts described above, a simplified procedure for assessing levee seismic vulnerability has been developed for the DWR Urban Levee Program, as presented below. This procedure may be refined during program implementation.

1. Estimate the peak horizontal acceleration (PHA) for the appropriate return period. Using an appropriate probabilistic seismic hazard maps (such as Figure 1), estimate the PHA at the subject levee site.
2. Evaluate the cyclic stress ratio (CSR). Using the CSR versus depth charts (Figure 3), estimate the CSR at the appropriate depths and for the corresponding locations adjacent to and below the levee (waterside, levee crest centerline or landside) for a given PHA at the site.
3. Using CSR information and site-specific information, perform Seed-Idriss-type simplified liquefaction triggering analyses to evaluate liquefaction potential in the liquefiable soils at the site using SPT-based procedures (e.g., Cetin et al, 2004) or CPT-based procedures (e.g., Moss et al., 2005) or other similar methods.
4. If the factor of safety against liquefaction (FS_l) is less than 1.0 for these soils, evaluate their undrained residual strengths using an appropriate relationship, such as the chart by Seed and Harder (1990) that relates penetration resistance (a measure of density) and undrained strength. For non-liquefiable soils, in some situations where the FS_l is close to but greater than 1.0 or significant softening is likely, assessment of seismically-induced strength degradation may be also be appropriate.
5. Using the appropriate seismic soil strengths (liquefied or non-liquefied, based on the results of step 4) and seepage pore water pressures, perform slope stability analyses of the levee section to evaluate K_y of potential critical sliding blocks.
6. Depending on the site-specific levee profile and subsurface conditions, choose a proper site model (softer, medium, or stiffer) and estimate the maximum seismic coefficient (K_{max}) of the above-identified sliding blocks from the PHA value and representative location of the considered block (Figure 4).
7. Compute the ratio between site-specific estimate of K_y / K_{max} for an appropriate return period and earthquake magnitude and estimate the Newmark displacement of the sliding block by using the displacement versus K_y / K_{max} chart (Figure 5).
8. Evaluate the magnitude of freeboard lost due to displacement. For each foot of horizontal movement, assume an appropriate amount of vertical displacement (typically a ratio on the order of 0.7 foot vertical displacement for every 1 foot of horizontal displacement, depending on the location and depth of the failure surface under consideration).
9. Assess the post-seismic flood protection ability of the levee using the following Table.

Four-Tiered Levee Seismic Vulnerability Classification System

Amount Of Deformation	Significant Damage to Internal Structures (e.g., Cutoff Walls)	Remaining Freeboard for Post-Seismic Evaluation (2-Year Flood Water Surface Elevation)	Post-Seismic Flood Protection Ability
<1'	No	>1'	Probably Uncompromised
1' to 3'	Possibly	>1'	Possibly Compromised
3' to 10'	Likely if existing	None	Likely Compromised
Unlimited (flow slide condition)	Yes	None	Compromised

If the internal configuration of the levee is critical to its performance and more vulnerable to even small displacements than typical levees (e.g. cutoff walls), then the analyst should consider increasing the seismic vulnerability rating to reflect the uncertainty of post-seismic performance.

EXAMPLE RESULTS

The following presents an example of results from one of the Urban Levee program study sites. At this location, the 500-year PHA is estimated to be 0.17g. A layer located in the foundation approximately 50 feet below the crest is comprised of silty sand with a corrected clean-sand equivalent SPT blowcount ($(N_1)_{60\text{-cs}}$) of 3 to 7. The induced cyclic stress ratio is on the order of 0.8 and is sufficient to trigger liquefaction. When liquefied, the sand has an estimated strength of 100 psf or less, leading to estimates of K_y of >0.085 on the landside and 0 on the waterside. Deformation for a Magnitude 7 earthquake is estimated to be less than 1 foot on the landside and unconstrained on the waterside. As indicated in the following table, this levee will likely be compromised during a seismic event and unable to provide post-earthquake flood protection.

500 Yr Return Period	Land Side	Water Side
Scenario	**Deep Failure**	**Deep Failure**
Post-Liquefaction Static Slope Stability Factor of Safety	2.08	0.71
FS Pseudo-static Analysis (K = 0.085)	1.12	Flow Slide - Calculation Not Needed
PGA	0.17	0.17
Kmax	0.145	0.145
Ky	>0.085	**0**
Ky/Kmax	**> 0.5**	**0**
Newmark Displacement (feet)	< 1'	Potentially Unlimited
Seismic Vulnerability Class	**Probably Uncompromised**	**Compromised**

Because estimated displacements are small for values of K_y / K_{max} > 0.5, in the DWR Urban Levee study, to avoid unnecessary K_y computations, a trial value of horizontal acceleration equal to one-half of the PHA is first evaluated in the slope stability calculations. If the Pseudo-Static Factor of Safety for this case is greater than 1, confirming that K_y / $K_{max} \geq 0.5$, then no further slope stability computations are considered necessary; Estimated displacements will be small.

CONCLUSION

Often levee seismic performance has not been evaluated because many believe that the probability of having a concurrent flood and earthquake are remote. Unfortunately, this may not be the controlling condition. An earthquake that damages a flood control system can be impacted by a relatively minor flood afterward, before repairs can be made, leading to significant flooding. The approach described in this paper allows engineers to classify levee seismic vulnerability, based on induced deformation and post-seismic flood protection capabilities. This method is currently being used to evaluate over 300 miles of urban levees in the Central Valley of California. Initial results indicate that levees in many locations are vulnerable to earthquakes and will likely not be able to provide flood protection after an event. As engineers and policy makers grapple with the implications of these analyses and make decisions about how to respond, further actions ranging from additional site investigations, more sophisticated seismic analyses, risk analyses, remedial designs, enhanced emergency response procedures and land-use policy decisions may all be considered.

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