# 30% CONCEPTUAL DESIGN REPORT:

# WASTEWATER STORAGE & RECLAMATION

## OCCIDENTAL COUNTY SANITATION DISTRICT

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**Prepared By:**

Brelje & Race Consulting Engineers
475 Aviation Blvd., Suite 120
Santa Rosa, CA 95403
v. 707.576.1322 f. 707.576.0469

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# LIST OF ABBREVIATIONS

| | |
|---|---|
| ADWF | Average Dry Weather Flow |
| CARB | California Air Resources Board |
| CDF | State of California Department of Forestry |
| CDO | Cease and Desist Order |
| CY | cubic yards |
| DSOD | State of California Division of Safety of Dams |
| ESD | Equivalent Single Family Dwelling Unit |
| gpd | gallons per day |
| gpm | gallons per minute |
| MG | million gallons |
| MGD | million gallons per day |
| NOA | Naturally Occurring Asbestos |
| NPDES | National Pollution Discharge Elimination System |
| psi | pounds per square inch |
| RWP | Recycled Water Plan |
| SCWA | Sonoma County Water Agency |
| TSO | Time Schedule Order |
| USACE | US Army Corps of Engineers |
| WDR | Waste Discharge Requirements |
| WTF | Wastewater Treatment Facility |

# 1. INTRODUCTION

## 1.1 BACKGROUND

The Occidental County Sanitation District (District) provides wastewater collection, treatment, storage and discharge services to the unincorporated community of Occidental, in western Sonoma County, California. The District treats wastewater from approximately 283 Equivalent Single Family Dwelling Units (ESDs). The community generates approximately 19,000 gallons per day (gpd) of wastewater under dry weather flow conditions. The Sonoma County Water Agency (SCWA) operates and manages the wastewater treatment facilities (WTF) under contract with the District.

The WTF is permitted by the North Coast Regional Water Quality Control Board (Regional Board) under National Pollution Discharge Elimination System (NPDES) Permit No. CA0023051, Order No. 93-42, dated September 17, 2007. The NPDES permit allows the WTF to discharge secondary treated effluent to Dutch Bill Creek, a tributary to the Russian River, from October 1 through May 14. During the discharge season (October 1 through May 14, flow conditions permitting) the effluent is discharged into a tributary to Dutch Bill Creek or is used for irrigation purposes. When discharges are prohibited (May 15 through September 30), the water is used solely for irrigation purposes.

As a result of threatened or continued discharge violations, the Regional Board adopted Cease and Desist Order (CDO) No. R1-2005-0085 and Time Schedule Order (TSO) R1-2005-0086 in October 2005. The CDO and TSO required the District to complete the construction of the Occidental Collection System upgrade project, cease and desist from threatening to discharge in violation of the WDRs, and come into compliance with the WDRs and the Water Quality Control Plan for the North Coast Region (Basin Plan) by addressing storage capacity, treatment and operation of the WTF.

In 2008 the District replaced 4,000 feet of sewer mains and public laterals, thereby reducing long-standing historical inflow and infiltration problems in the collection system. Continuing its efforts to comply with the WDRs and CDO, the District is pursuing a project to construct storage for its treated effluent. The District desires to construct a reservoir with capacity sufficient to allow it to cease discharging to surface water. Effluent would be stored during the rainy months for use in irrigating agricultural lands during the growing season. Concurrently, the irrigation area would be expanded to accommodate the water generated during a high rainfall year.

## 1.2 PURPOSE AND SCOPE

- Improve existing storage and/or add new storage, making it sufficient to store all of the WTF effluent through the rainy season.
- Expand the existing irrigation system to a size that is sufficient to dispose of the entire year's WTF effluent through the irrigation season.

The overall purpose of this report is to develop three reservoir alternatives and identify sufficient irrigation area for reclamation, analyze the advantages, disadvantages, and life-cycle cost differences between alternatives and develop a 30% conceptual design of a preferred alternative. The reservoir alternatives have been defined as follows:

- Modification of existing on-stream reservoir with a stream by-pass and cover. Construction of a supplementary pond. (Alternative 1)
- Construction of larger reservoir in same location as existing reservoir, with stream by-pass and cover. (Alternative 2)
- Construction of one large off stream reservoir with cover. Abandonment of the existing reservoir. (Alternative 3)

This report includes conceptual design criteria, analysis of storage capacity and irrigation area requirements, summary of applicable regulatory limitations, geotechnical findings and recommendations, a conceptual design, a construction cost estimate and identification of Right-of-Way issues. Supporting documents in the Appendices include engineering analysis of storage capacity and irrigation area requirements, a summary of applicable regulatory limitations, geotechnical report, summary data and figures depicting pond options, and laboratory reports from soils tests for asbestos.

## 1.3 OBJECTIVES AND GOALS

The objectives of this report are as follows.

- Establishment of valid design criteria
- Identification of regulatory limitations
- Exploration of feasible pond construction site(s) and configuration(s) which would comply with Division of Safety of Dams (DSOD) regulations through construction of dam height less than 25 feet.
- Determination of probable costs
- Identification of preferred reservoir site and associated irrigation areas
- Development of 30% conceptual design

The goal of this report is to provide a 30% conceptual design of wastewater storage and irrigation facilities, to form as basis for environmental impact analysis and for subsequent development into construction documents.

# 2. EXISTING FACILITIES

## 2.1 GENERAL DESCRIPTION

The District facilities include a collection system, secondary treatment facilities, storage in a holding pond (Graham's Pond), and discharge and irrigation facilities. Figure 1 shows the locations of the existing sewage lift station and force main, WTF, storage pond, irrigation areas and discharge to surface water.

## 2.2 DESCRIPTION OF FACILITIES

### 2.2.1 Service Area

The District serves the unincorporated community of Occidental, located in western Sonoma County at the junction of Bohemian Highway, Graton Road, Occidental Road and Coleman Valley Road. There are approximately 45 dwelling units, approximately 30 business and commercial establishments, and two churches in the town. Several restaurants are the primary base of the town's economy.

The District has generated an Average Dry Weather Flow (ADWF) in the range of 14,000 gpd to 19,000 gpd for the past fifteen years.

### 2.2.2 Collection System

The District's collection system consists of approximately 7,500 feet of gravity mains and associated manholes. A project completed in 2008 that replaced 4,000 feet of sewer mains and public laterals reduced long-standing historical inflow and infiltration problems in the collection system.

Wastewater from the Occidental collection system flows to a lift station adjacent to Dutch Bill Creek, approximately a quarter mile northerly of town. Three above-ground 20 horsepower (HP) positive displacement pumps lift the raw sewage through a 4-inch force main to the wastewater treatment plant. A diesel generator provides back-up power during electricity outages.

### 2.2.3 Treatment Plant

The District's WTF is located easterly of town on approximately 1.5 acres owned by the Order of Druids. A Grant of Easement was executed by the Druids Order in 1968 to the District for areas including the treatment plant site and the influent and effluent pipeline alignments. Secondary treatment is provided at the WTF, including aeration, settling, disinfection with chlorine, followed by dechlorination and pH adjustment.

### 2.2.4 Storage

Disinfected effluent flows by gravity from the chlorine contact tanks through a 6-inch main to storage in Graham's Pond, located approximately one quarter mile northeasterly of the wastewater treatment plant. The District uses the privately-owned reservoir under an agreement with the land-owners.

The pond, which dams a tributary to Dutch Bill Creek, was originally built as a farm pond. There are no known records of its age or method of construction. The pond has a capacity of approximately 8.7 MG. Appropriative water rights were established in 1960 and 1970, for a combined maximum storage of 31 acre-feet of surface water at the site. Each Water Right allows for a maximum of 14 acre-feet to be withdrawn per year, for use in fire-fighting, stock watering and irrigation.

Surface water from the approximately 68-acre tributary drainage area flows into the pond during the rainy season. An estimated 25 to 100 MG of surface water flows into the pond each year depending upon rainfall conditions. For comparison, the annual treatment plant effluent volume discharged to Graham's Pond has averaged approximately 11 MG over the past ten years.

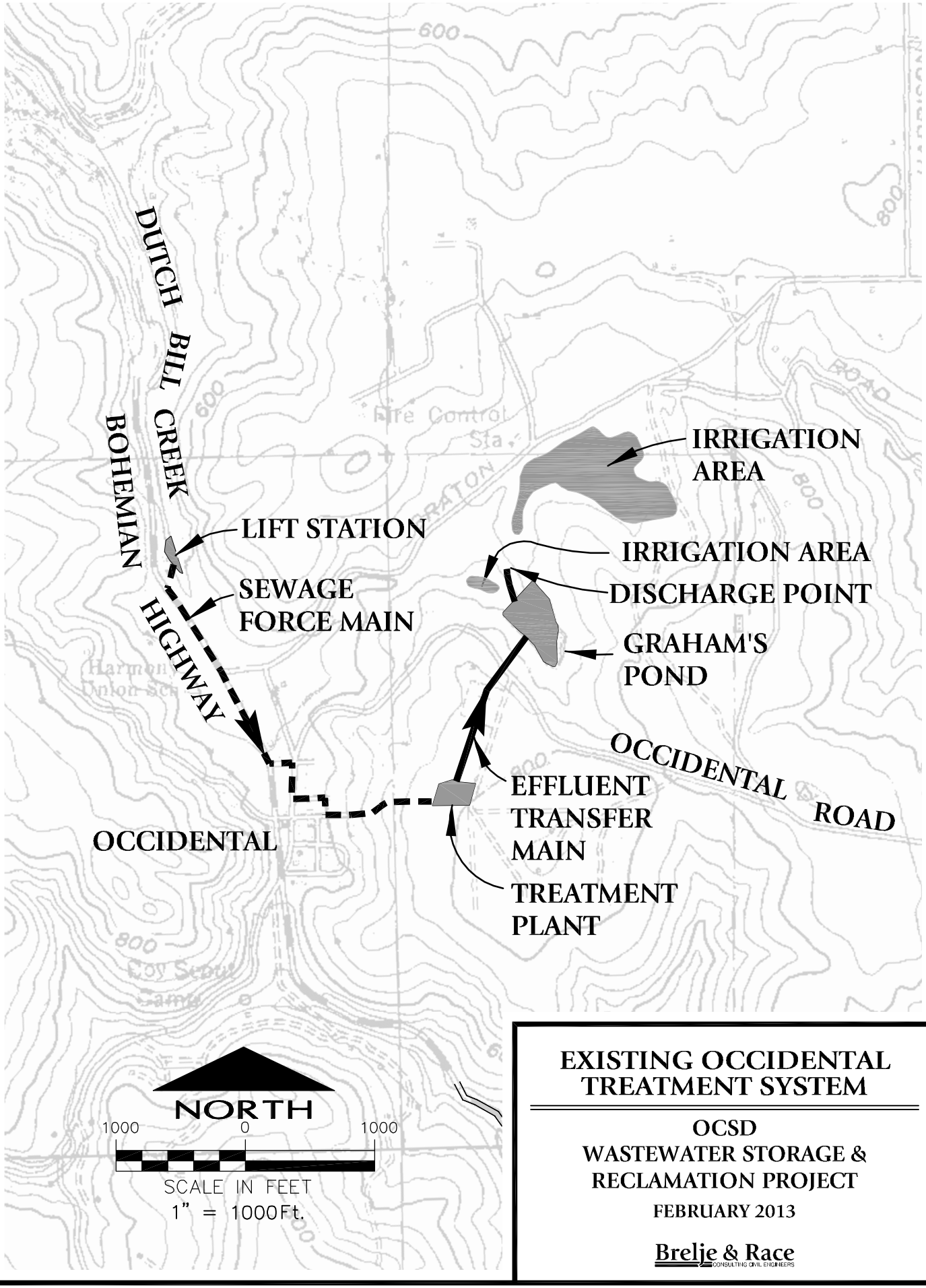
The Regional Board considers Graham's Pond to be "waters of the state" because it impounds a tributary to Dutch Bill Creek. Any discharge to "waters of the state" is required to meet Regional Board requirements for surface water discharge. Numerous discharges of District effluent to the pond have not met the District permit requirements for surface water discharge, and the Regional Board has cited these discharges as permit violations. The Regional Board has issued a Cease and Desist order against the District, in part on the basis of these violations. Additionally, discharge to Graham's Pond occurs year round, in conflict with the Basin Plan's discharge prohibition period from May 15 to September 30.

### 2.2.5 Discharge to Dutch Bill Creek

The District's NPDES permit allows for discharge to Dutch Bill Creek between October 1 and May 14, at not more than 1 percent of the flow in Dutch Bill Creek as measured at Camp Meeker. Discharge from Graham's Pond flows into the tributary to Dutch Bill Creek on which the pond is constructed. Due to the significant volume of surface water that flows into the pond, the District is permitted to discharge estimated surface water flows in addition to the effluent flow of up to 1 percent of Dutch Bill Creek flow.

### 2.2.6 Irrigation System

Water is pumped from Graham's Pond to irrigation on pasture located northeasterly of the pond on the Loades property. A small pump house is located below the dam. During the dry season, water is withdrawn from Graham's Pond for irrigation on approximately 8 acres of adjoining pasture. Irrigation water is pumped into piping mounted on the ground surface and discharged through spray heads. Irrigation operations are managed by District staff.




02-12-13 kifilzghi \3619\dwg\3619 02\3619.02-EXHIBIT-Treatment System.dwg TAB: FIGURE 1 - 8.5X11 1000SC

FIGURE 1

# 3. PROJECT AREA

## 3.1 BOUNDARIES AND TOPOGRAPHY

The project area, shown on Figure 2, consists of lands belonging to Loades, Colella, and Briggs. The terrain is comprised primarily of moderately sloped lands adjoining Graton Road, with steep hillside running from southwest to northeast above Graham's Pond. Elevations within the project area range from approximately 600 feet above sea level along Graton Road to approximately 850 feet above sea level at the southeasterly corner of the Loades property. Areas with steeper slopes are generally forested, while gentler sloping areas are pasture and grassland. The headwaters of Dutch Bill Creek flow south westerly across the Loades property, near Graton Road.

## 3.2 LAND USE

Lands within the project area are zoned Agricultural Resources (Briggs and Colella) and Diverse Agriculture (Loades). There are one or more residences on each of the properties. Much of the area is pasture or grassland, with wooded areas at the perimeters, generally on the steeper slopes. Cattle are put to graze on the Loades' and Briggs' properties.

## 3.3 GEOLOGY AND SEISMICITY

A geotechnical desk study was performed of the project area as the first phase of the geotechnical investigation. The desk study included a review of available geotechnical reports, maps and other data regarding the geology, tectonics, and seismicity of the overall site area to help facilitate identification of a preferred site and for use in an exploratory field investigation and analyses for that site. A copy of the Geotechnical Report is attached as Appendix C. A summary of the area's geology and seismicity follows.

The youngest lithified geologic strata in the area, the Wilson Grove Formation marine sandstone generally forms the higher ground surrounding the general site area and has been irregularly eroded in the central site area revealing underlying Jurassic basement formations comprising fault bounded Serpentinite matrix mélange, Knoxville Formation shale, Keratophyre tuff (volcanics) and Franciscan Complex mélange.

The Serpentinite forms the northeast side of a northwesterly draining valley containing most of the Loades parcel north and about half of the Loades parcel south, and including the northeast half of the existing Graham's Pond. The northeast half of the Loades parcel south is mostly underlain by the keratophyre tuff formation; the southwest side of the valley, including the southwest half of the existing Graham's Pond is underlain by Franciscan greywacke (sandstone) and serpentinite mélange.

The southern ends of both the Briggs parcel and the Colella parcel and the northern portion of the Loades north parcel along the flatter areas along Graton Road are underlain by alluvial deposits that would need to be assessed for liquefaction potential if chosen for further subsurface study. These alluvial deposits are generally described as undivided sand, gravel, silt and clay and in the site area occur as active stream channel units and small alluvial fans. These deposits approximately underlie the flatlands on either side of the Graton Road in the site area.

The Wilson Grove Formation is described as largely fine grained sandstone with variably abundant silt and some clay matrix; sandstone ranges from clean to saturated by silt and clay. Much bedrock has moderate permeability. Some bedrock and much mantle (near surface weathered material) is

significantly expansive. This unit approximately underlies a small portion of the Colella and Briggs parcels on the northwest end of the revised site area.

Keratophyre Tuff Unit is described as deeply weathered breccia in this area (deposited as extrusive volcanics / pyroclastic materials), most of which is firm, but most mantle is significantly expansive. This unit approximately underlies the southerly and easterly portion of the Loades parcels and southern portion of the Briggs property.

Franciscan Mélange Unit is described as sheared/brecciated sandstone and shale including some gouge (highly sheared fault zone material) and may contain hard blocks in greatly varying proportions (hand size to boulder size or larger), is largely unexpansive, but mantle can be unexpansive to extremely expansive. This unit approximately underlies the northwestern portion of the Loades property, including the southwest half of Graham's Pond. The Knoxville Formation shale Unit is described as largely mudstone and shale of which both bedrock and mantle are unexpansive to significantly expansive. This unit underlies a small portion of the Loades property.

### 3.3.1 Landslides

Three landslides are mapped in and adjoining the project area:

- A large southeast facing landslide which appears to encompass about half of the Colella parcel, and extends from the higher ground on the northwest side of the parcel to the alluvial soils along Graton Road on the southeast side of the parcel;
- A smaller but significant south facing landslide is shown on the southwest side of the Colella parcel (a northwest facing landslide is mapped immediately to the south but does not appear to impinge on the study area);
- A large northwest facing landslide is shown extending north from the northern portion of the Loades south parcel, extending down through a portion of the Loades north parcel to the alluvial flatland deposits.

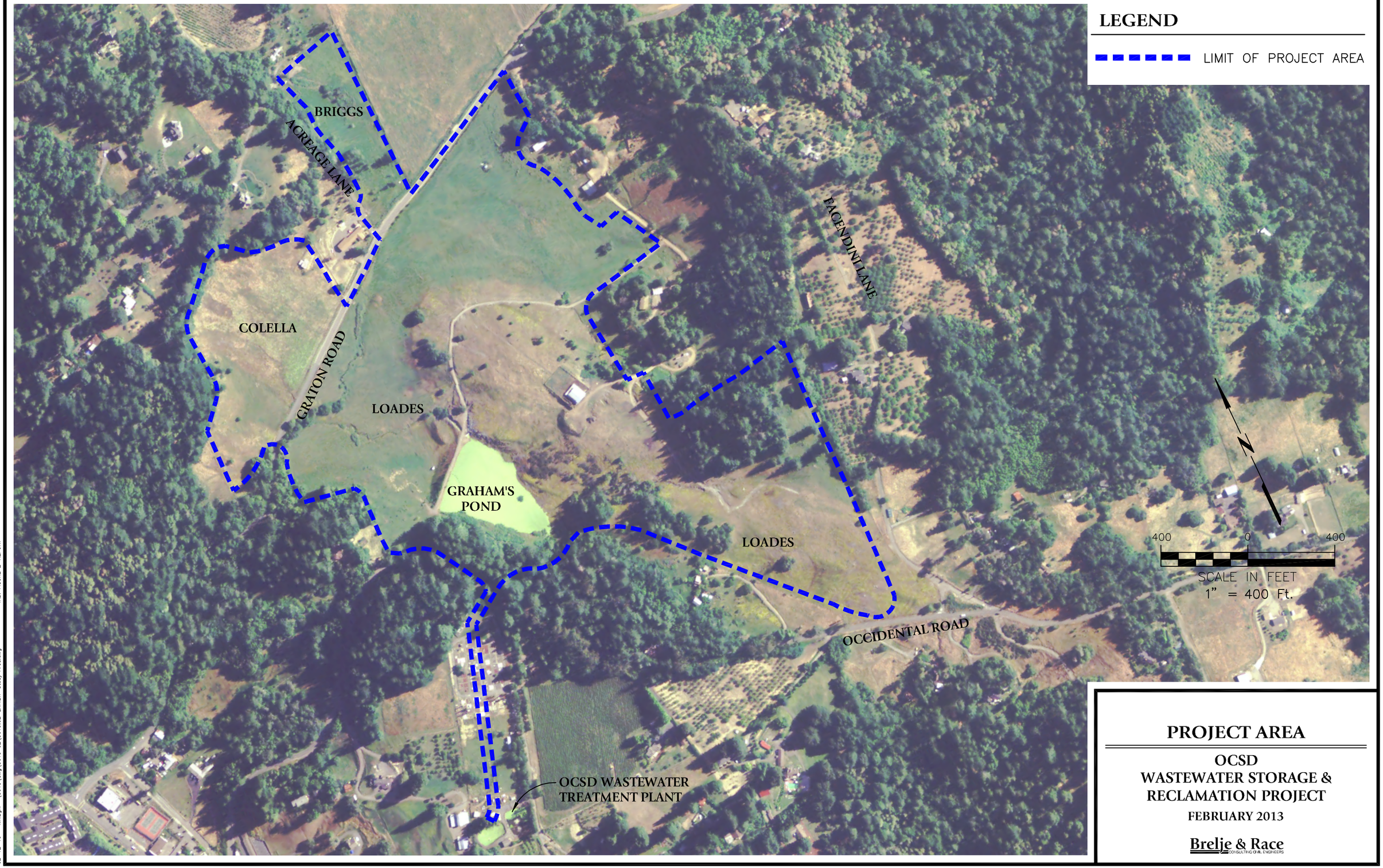
### 3.3.2 Site Area Faulting

The site is not located within a State-designated, Alquist-Priolo Earthquake Fault Zone where site-specific studies addressing the potential for surface fault rupture are required, and no known active faults traverse the site.

The closest expression of the San Andreas Fault is located approximately 7.6 miles to the west of the site. The closest expression of the Rodgers Creek Fault is located approximately 10.7 mile to the east northeast.

## 3.4 ENVIRONMENTAL RESOURCES

The study area is rural. The majority of the Loades and Briggs parcels have been subject to significant grazing. Primary plant communities in the study area include Mixed Evergreen Forest, Non-Native Grassland, and Willow and Blackberry Riparian. There are also serpentine areas and aquatic areas which includes Graham's pond, the headwaters of Dutch Bill Creek and several drainages. In addition, there are scattered, remnant chaparral areas including occasional coyote brush stands that are not included as a plant community in this document due to their sparse and patchy vegetation. The Loades parcel is known to support wetlands and the Briggs and Colella parcels exhibit wetland indicators. There are several special status plant and animal species that have been




## PROJECT AREA

OCSD
WASTEWATER STORAGE &
RECLAMATION PROJECT

FEBRUARY 2013

Brelje & Race
CONSULTING CIVIL ENGINEERS

FIGURE 2

02-12-13 kifizighi \3619\dwg\3619 02\3619.02-EXHIBIT-Study Area.dwg TAB: FIGURE 2-REPORT

confirmed to be on-site or could be supported by habitats occurring on-site. Environmental resources are discussed at length in the associated Environmental Information Document.

### 3.5 HISTORICAL AND ARCHEOLOGICAL RESOURCES

There are two previously recorded prehistoric archaeological sites in the study area. Prehistoric site CA-SON-367 is located on the Briggs parcel on the north side of Graton Road. The second site (P-49-2636) was recorded by Tom Origer & Associates in 1999. This site is on the Loades parcel, south of Graton Road. Both sites are shown on the map of environmental constraints, Figure 3. Construction within either site would require archaeological investigation. Irrigation on the sites, provided pipes are above ground, is acceptable.

# 4. DESIGN CRITERIA

## 4.1 GENERAL

Design elements include an effluent storage pond, recycled water irrigation area, and appurtenant piping and pumping facilities. Design criteria include capacity requirements, regulatory limitations, and engineering criteria.

This section of the report describes the development of design criteria.

## 4.2 DESIGN CRITERIA

### 4.2.1 Storage Pond and Irrigation Area Capacities

The design capacities for the reservoir and irrigation area were determined based upon projected future District wastewater generation in the 100-year recurrence peak rainfall year. A water balance model was developed to calculate the storage and irrigation requirements for varying wastewater generation and weather conditions. In order to avoid constructing as large a pond as would be required to accommodate direct rainfall, the District determined that it would construct a cover on the pond to exclude direct rainfall. A discussion of the development of water balance parameters may be found in Appendix A, Technical Memorandum No. 1, Water Balance Parameters and Outputs. Design criteria for storage and irrigation capacities are presented in Table 1.

**Table 1: Design Criteria for Storage and Irrigation Capacities**

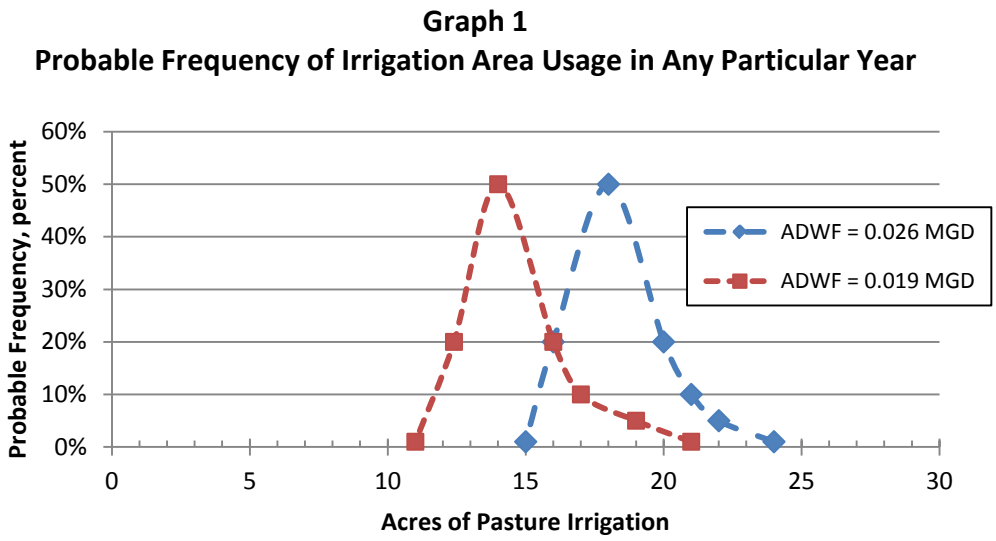
| **Projected ADWF 26,000 gpd** | | **Current ADWF 19,000 gpd** | |
|---|---|---|---|
| Required Storage Capacity, MG | Required Irrigation Area at 27" per Year, acres | Required Storage Capacity, MG | Required Irrigation Area at 27" per Year, acres |
| 12.5 | 24 | 11.0 | 21 |

MG - million gallons
The design criteria are based on the 100-year recurrence total annual rainfall

The required storage capacity is 12.5 million gallons (MG). The required irrigation area is 24 acres of pasture that uses 27" of irrigation water per year. If lands are irrigated which utilize more or less irrigation water, the requisite area would be increased or decreased in proportion to the relative water usage.

Actual use of the storage capacity and irrigation area will be less than the design capacity in all years except the 100-year recurrence peak rainfall year, and only after the District has attained the projected future wastewater ADWF. Projected use of the storage and irrigation area for various rainfall and ADWF conditions are shown in Appendix A, page 5, Table 1.

While the District would want to construct a pond with the capacity to meet projected future storage needs, irrigation areas would be developed to meet current needs and expanded as needed in the future. Graph No. 1 shows the probable frequency of using a particular number of acres of irrigation land in any given year.




As can be seen in Graph No. 1, at the current ADWF of 0.019 MGD, the District would expect to need an irrigation area of 17 acres in ten percent of the years. Put another way, in 90% of the years, an area of 17 acres would be sufficient. The District may want to develop an irrigation system that is adequate for 90% or 95% of weather years, and to be prepared to irrigate other areas with portable irrigation equipment when needed. Also, in dry weather years the District would be able to provide only a percentage of the water that it would provide in average and wet years.

### 4.2.2 Storage Pond Configuration

The storage pond should be configured to allow access for operations and maintenance, with an access road and a perimeter road around the entire top-of-dike circumference. Roads should be designed to accommodate service loads necessary for operation and maintenance. Ramps to and from perimeter dike roadways should be designed with slopes suitable for two-wheel drive access, no greater than 15%.

Pond side slopes, inside and out, must be limited to an angle that will be stable. To be conservative, the preliminary design criteria for the side slopes, both inside and outside the pond, has been set at 3:1. Site specific analysis, particularly geotechnical analysis, may allow for steeper slopes as the design develops.

### 4.2.3 Storage Pond Lining and Cover

The storage pond will be constructed with a cover to prevent rainwater from entering the pond. The cover will be constructed following current standard practice for pond covers. The cover will be fabricated from a synthetic membrane material with high ultraviolet light resistance and tear strength; the recommended material is Hypalon®. The cover will be constructed to float on the water, with folds that will open as the water level goes down and close as the water level rises. The folds will drain to one or more sumps in the cover. Pumps in the sumps will pump accumulated rainwater to infiltration areas outside of the pond constructed for this function.

The pond must have a liner that controls seepage in a manner that protects the underlying groundwater from degradation, in order to be permitted by the Regional Board. Pond lining may be accomplished by: (1) a clay liner constructed to a controlled permeability, coupled with analysis of the potential seepage to groundwater and water quality impacts, or (2) a synthetic membrane liner designed to prevent any seepage from the pond. While the cost of the construction of the compacted clay liner may be lower than for a synthetic liner, there is uncertainty whether an analysis of water quality impacts would demonstrate protection of groundwater. For this reason, the 30% conceptual design and budget estimate will be based on construction of a synthetic liner.

### 4.2.4 Regulatory Limitations & Permitting Requirements

A review of State and Federal regulations was conducted, and anticipated regulatory limitations and permitting requirements for the reservoir and irrigation area site selection, construction and operation were identified. Please refer to Appendix B, Technical Memorandum No. 2, Regulatory Limitations and Permitting Requirements, for this review and for a summary list of the requirements of governmental agencies having jurisdiction to approve portions of the Project. Design criteria that were developed from the regulatory review are listed in Table 2.

**Table 2: Design Criteria drawn from Regulatory Limitations**

| Regulation / Enforcing Agency | Design Criteria/Limitation |
|---|---|
| California Water Code/ Division of Safety of Dams (DSOD) | Design will target a difference in elevation between the lowest outside toe of bank and the maximum water level of less than 25 feet, in compliance with DSOD jurisdictional limits. |
| | If no satisfactory sites exist to construct a pond that would be outside jurisdiction, the District may decide to evaluate jurisdictional options. |
| | Construction on the existing dam or within Graham's Pond may fall under DSOD jurisdiction. |
| Title 22/Regional Board | Pond and irrigation sites will provide a minimum 100 foot set-back from any domestic water supply well. |
| Anticipated Waste Discharge Requirements (WDRs)/ Regional Board | The District will have to demonstrate that a new reservoir will not cause degradation of groundwater. Options are to:<br>• Construct a synthetic liner to prevent seepage to groundwater, or<br>• Design a compacted clay liner, characterize the potential seepage, and demonstrate that potential seepage will not degrade groundwater. |
| | If the District selects continued use of Graham's Pond, stream flow will need to be routed around the reservoir. |
| | Reservoir will be designed with a maximum operating level that is three feet below the top of dike, assuring that even in emergency conditions a minimum freeboard of two feet will be maintained. |
| | New irrigation areas will be configured to include a 100-foot setback from surface waters or developed in a manner to prevent or minimize the potential for runoff discharging to surface water. |
| California Air Resources Board (CARB) | If a proposed pond site would involve construction within an area designated as ultramafic, or an area with apparent ultramafic rocks, representative samples of the rock will be tested for asbestos. If asbestos is found, a project response will be developed to minimize disturbance of areas with ultramafic rock and to meet applicable regulations for air quality protection and waste disposal. |

### 4.2.5 Transfer Piping

Piping for transfer to a new pond location would be sized to convey the projected maximum flow from the WTF. Flow from the WTF is currently limited to approximately 140 gpm (0.20 MGD), the allowable pump rate through the chlorine contact basins. The maximum daily transfer since the sewer project was completed has been 0.16 MGD, in February 2009. The District has no plans to modify the contact basins or increase the disinfection pump flow rate.

The existing main from the WTF to Graham's Pond is 6-inch diameter. The main was constructed of PVC, in 1985. The pressure class of the PVC piping is not known. The lower portion of the transfer main, on the steep slope into Graham's Pond, is ductile iron pipe. Any extension of the transfer main to a new pond site would be designed to convey flows of 0.2 MGD. While a four-inch or six-inch main would be adequate for conveying these flows, a six-inch main is recommended for future flexibility. For 30% conceptual design, it will be assumed that the existing piping has adequate pressure rating for anticipated higher pressures for transfer to a pond at a higher elevation. This assumption will need to be verified in the next phase of design.

For pond alternatives located at elevations higher than the WTF, a pump station would be constructed at the WTF. The single speed pump rate would pump at a slightly greater flowrate than the pump rate through the chlorine contact basins. A redundant pump would be provided. The existing arrangement could be retained for testing real-time pH values and returning effluent that does not meet discharge requirements to the settling pond. It would no longer be necessary to dechlorinate the effluent once the District became a no-surface-water-discharge system.

### 4.2.6 Irrigation System

Irrigation pumping and conveyance infrastructure would be sized to meet peak irrigation demands. Projected peak irrigation demands have been estimated based upon the volume of water to be used and on typical irrigation patterns for pasture and for vineyards in Sonoma County. These values are shown in Table 3.

**Table 3: Typical Irrigation Demands**

| Crop | Pasture, Hay, Fodder Crops | Vineyard |
|---|---|---|
| Annual usage | 27" | 3" – 5" |
| Typical Irrigation Mode | 7 sectors, each watered once a week @ 0.2 in/hr/irrigated acre | Vineyard in 3 sectors, each watered at 12-24 gpm/irrigated acre |
| Overall Irrigation Rate | 13 gpm/acre | 4-8 gpm/acre |
| Minimum Pressure at Turn-Out | 60 psi | 60 psi |
| Peak Irrigation Demand * | 310 gpm | 640-1,280 gpm |

*For design criteria pasture area of 24 acres, or for vineyard area of 160 acres. Actual peak demand would depend upon the mix of land uses in the irrigation area.

For conceptual design, pump horsepower and piping diameters would be sized to provide flows and pressure listed in Table 3 for pasture or hay fields. Three-phase power would be provided for the pump station, including new service if needed. A redundant pump would not be provided for irrigation. Distribution piping would be 6 and 4-inch diameter buried PVC, with surface connections for portable aluminum latch-pipe laterals.

# 5. METHODOLOGY FOR SITE EVALUATION

## 5.1 GENERAL

After development of design criteria, the next step in the project consisted of identifying and screening potential pond sites. Potential sites were defined as follows:

- Modification of existing on-stream reservoir (Graham's Pond) with a stream by-pass, liner and cover. Construction of a supplementary pond. (Alternative 1)
- Construction of larger reservoir in same location as existing reservoir (Graham's Pond), with stream by-pass, liner and cover. (Alternative 2)
- Construction of one large off stream reservoir with cover and liner. Abandonment of the existing reservoir. (Alternative 3)

The potential storage pond and irrigation areas were evaluated for topographical, geological and geographical limitations, environmental impacts, land-owner and neighborhood concerns, and regulatory requirements. The methodology for evaluating sites is described below. Figure 3 presents a topographical map of the project area showing physical constraints that were revealed during site evaluation. Section 6 presents a review of the sites that were identified.

## 5.2 TOPOGRAPHY

The project area is generally hilly, and most flat areas have one or more streams and/or wetlands running through them. This combination of topography and surface water alignments makes siting of a 12.5 MG storage pond a significant engineering challenge, aside from other considerations. Because of the topographic challenges, the methodology for identifying potential sites focused first on a general consideration of topographic limitations. The topographic map of the study area was reviewed for areas with gentle slopes that appeared large enough for siting a storage pond with maximum dam height of 25 feet.

Figure 3 shows the areas identified as having potential for siting a pond during this phase of work. A pond configuration was developed for each site, and illustrated using AutoCAD Civil 3D. The sites with potential for storage pond reservoir construction were then reviewed for constructability and other considerations, according to the criteria discussed in the following sections.

## 5.3 GEOLOGY & GEOGRAPHY

### 5.3.1 Geotechnical Concerns

A desktop study and site surface review were performed for the identified potential storage pond sites. The findings are presented in the Geotechnical Report, Appendix C, and discussed in Section 3, under Geology and Seismicity.

The desktop study and site surface review revealed a band of ultramafic rock (serpentinite mélange) running from southeast to northwest through the Loades property. The generalized extent of the serpentinite band, as shown on the State Geologic Map, is shown on Figure 3. Figure 3 also shows a line indicating the field-determined easterly boundary of the ultramafic band, in the southeast portion of the project area. Construction in areas with mapped ultramafic rock must include measures to control dust in compliance with California Air Resources Board (CARB) regulations

(Technical Memorandum No. 2, Appendix B), because of the possibility of asbestos in ultramafic rock. The potential for asbestos existing in the mapped areas of ultramafic rock is a consideration in site evaluation, and samples of the soil/rock would be tested as part of selection of a preferred alternative.

Any materials removed from the site would be subject to hazardous waste disposal regulations if they contained friable asbestos at concentrations greater than 1%.

### 5.3.2 Relative Elevation

A storage pond site at a lower elevation than the WTF would not require pumping for transfer flows. If irrigation areas are at a lower elevation than the storage pond, irrigation pumping can be minimized.

### 5.3.3 Current Land Use

Existing high-value land uses, such as home sites or areas currently used for irrigation of the District's recycled water, were considerations in selection of alternative locations.

### 5.3.4 Distances and Routes from Point of Source and Point of Use

Construction of the new storage pond will include piping to convey transfer flows to the pond and irrigation flows from the pond. A site closer to the WTF and/or irrigation areas would require less pipe construction and ground disturbance.

### 5.3.5 Access

Site evaluation included exploration of issues associated with access. Preliminary access road alignments were established and evaluated.

### 5.3.6 Safety

Potential route(s) for flooding in event of a dike failure were considered, in particular potential threat to any existing homes in the floodway path.

### 5.3.7 Irrigation Area Selection

In delimiting potential irrigation areas, areas in proximity to and down-gradient from the storage pond site would be evaluated preferentially. Lands with crops, such as pasture and fodder crops that use large amount of irrigation water would be preferred to areas with low-water using crops, such as vineyards. To minimize irrigation run-off, irrigation areas would generally be restricted to lands with slopes of 15% or less. Areas with slopes between 15% and 20% would be considered for irrigation after exceptionally wet winters; such areas would be irrigated only every 10 to 20 years, on average. Areas on the Loades parcel that are currently irrigated would be recommended for continued irrigation, unless they were used for construction of a storage reservoir.

## 5.4 ENVIRONMENTAL CONSIDERATIONS

Potential pond locations were screened for likely environmental obstacles to pond construction and irrigation area development. Permitting requirements by the US Army Corps of Engineers

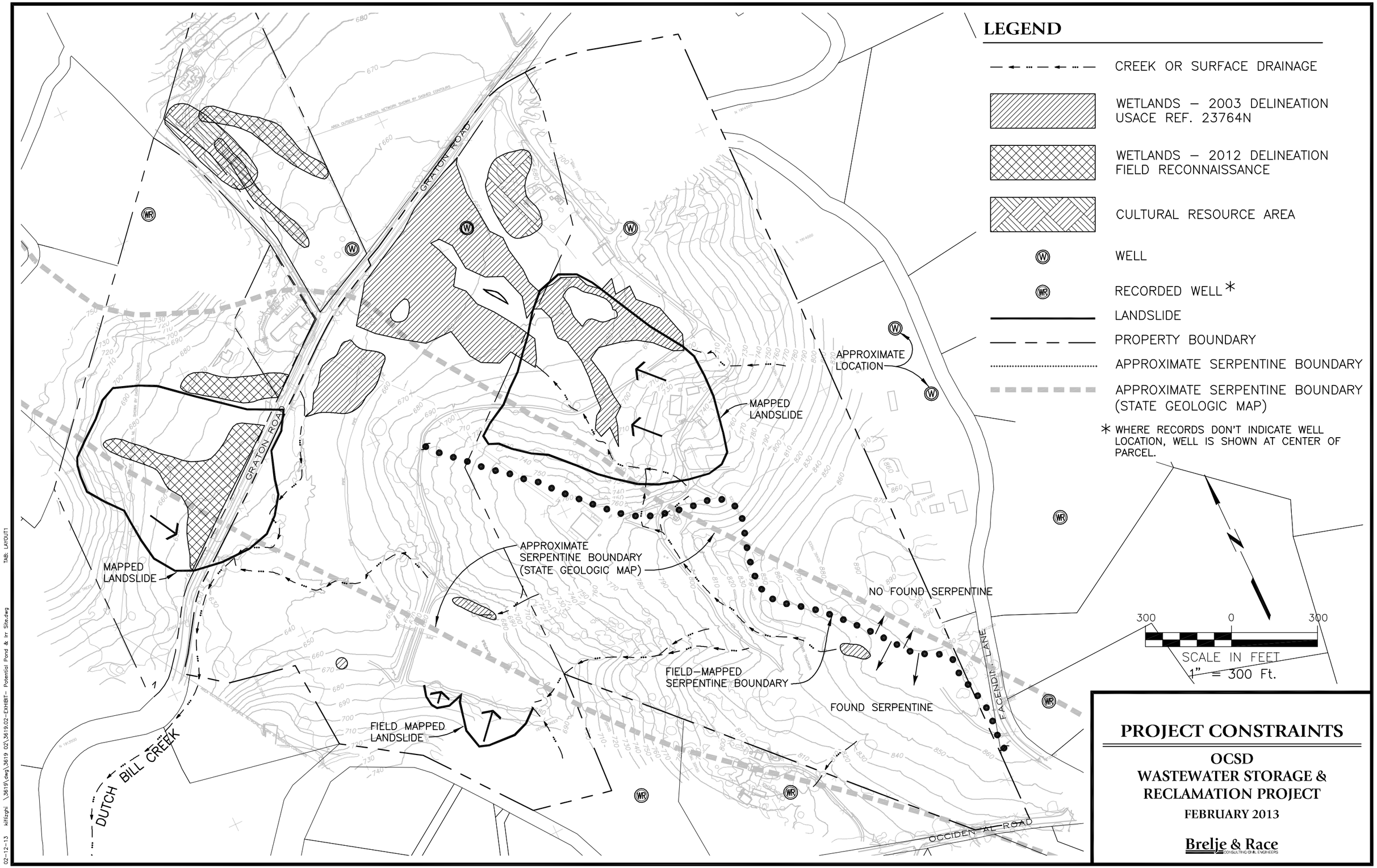



FIGURE 3

02-12-13 kifizghi \3619\dwg\3619 02\3619.02-EXHIBIT- Potential Pond & Irr Site.dwg TAB: LAYOUT1

(USACE), the Regional Board, and the California Department of Fish and Game would apply to areas with wetlands and/or waters of the State and to areas that provide habitat for protected species. Sites near water courses would be more likely to have issues associated with wetlands, surface water impacts, endangered species, potential cultural resources and flooding, and resultant impacts to project costs.

Potential pond and irrigation sites will be evaluated for impacts to water courses, including apparent and previously delineated wetlands. Any required mitigation for wetland loss would add significantly to the costs of pond construction. The Regional Board does not permit irrigation with recycled water to enter surface waters, as discussed in Section 4.

## 5.5 LAND-OWNER AND NEIGHBORHOOD CONCERNS

### 5.5.1 Visual Impacts

A consideration that has potential to be significant for any site is visual impacts. The reservoir will appear as an engineered dike if seen from a similar or lower elevation or, if seen from a higher elevation, as a dike surrounding an area covered by a synthetic membrane. A site that had less opportunity for off-site views and/or provided existing or potential trees as visual barriers would be preferred to a site with significant offsite visibility.

### 5.5.2 Land-Owner Concerns

There is a preference for siting on the Loades property, because the District has an existing agreement for use of areas within the Loades property and the owners have expressed an acceptance that the District needs a new storage pond and that their property is the preferred location for a new pond.

Because the Loades would like to have full use of Graham's Pond restored to them, the ability to construct a new pond with the design capacity would be viewed positively in screening potential sites. See also discussion of Water Rights, below.

### 5.5.3 Surrounding Land Use

Construction in proximity to residences may trigger concerns from homeowners about health issues, impacts to land values, odors, and noise, light and water pollution. Sites further from residences would avoid these issues. Well construction records kept by the State Department of Water Rights were reviewed and neighboring parcels with known wells are indicated on Figure 3.

## 5.6 LEGAL REQUIREMENTS

### 5.6.1 Easements

The need for easements for construction and/or operation of a storage pond or irrigation area at each potential site will be evaluated.

### 5.6.2 Utility Locations/Conflicts

Existing utility locations will be reviewed for each potential pond site to determine whether conflicts exist. Also, utility requirements, specifically power for irrigation pumps, will be evaluated.

### 5.6.3 Water Rights

The landowner on the Loades property holds two appropriative Water Rights (Applications Nos. A018736 and A019515), the first for 14 acre-feet of storage and withdrawal per year, and the second for 17 acre-feet of storage and 14 acre-feet withdrawal per year, at Graham's Pond. The water may be diverted between October 1 and May 31 of the succeeding year. If Graham's Pond were converted into a covered effluent storage pond, the landowner would have no capacity to store the water diverted under the Water Rights. There would be some risk of losing the Water Rights.

# 6. POTENTIAL PROJECT SITES

Potential pond sites include the existing Graham's Pond site, and five other sites in the Project area that were identified as having potential for pond construction. Figure 4 shows the potential pond sites, along with the project constraints discussed in Section 5, on a topographical map of the study area. This Section presents a review of each potential pond site, followed by a discussion of potential irrigation areas. Appendix D includes figures depicting pond configurations that were developed and rejected in favor of the configurations discussed herein.

## 6.1 GRAHAM'S POND: ALTERNATIVES 1 AND 2

Alternatives 1 and 2 have been defined as follows:

Alternative 1: Modification of the existing 8.5 MG on-stream reservoir with a stream by-pass, liner and cover. Construction of a second pond for total design capacity of 12.5 MG.

Alternative 2: Construction of 12.5 MG reservoir in same location as the existing reservoir, with stream by-pass, liner and cover.

### 6.1.1 Existing Dam

The existing dam may be within DSOD jurisdiction (Technical Memorandum No. 2, page 2). DSOD consultation would be required for any modification to the pond, including modification of the spillway, installation of liner and cover, or construction of a stream by-pass. Since there are no records of construction, it is not known whether the existing dam complies with DSOD standards. Since DSOD would require a demonstration that the existing dam meets DSOD construction standards, geotechnical exploration of the dam soil and underlying rock would be required. It is possible that it would be found that the dam did not meet DSOD standards. If this were the case, DSOD could require reconstruction of the dam. Costs for testing the dam structure and submitting the findings to DSOD might be better applied in demolition of the existing dam and construction of a dam to meet current standards. Cost estimates presented in this report for modifications to Graham's Pond are based upon the assumption that no tests would be performed, and the dam would be demolished. The new dam would not necessarily have to be within DSOD jurisdiction.

The current capacity of Graham's Pond was determined using District survey information and AutoCAD Civil 3D to be approximately 8.7 million gallons (MG), when the water level is at the invert of the spillway. If the dam has to be reconstructed, it would be most cost-effective to construct the reconfigured pond with a 12.5 MG capacity (Alternative 2), rather than construct a new pond with the existing capacity, plus a second pond to bring the total storage capacity to 12.5 MG (Alternative 1). The Alternative 1 construction costs associated with demolition and reconstruction of the Graham's Pond dam, which were not anticipated at the time that the pond alternatives were defined, plus construction of a second pond, would significantly exceed construction costs for a single pond under Alternative 2.

### 6.1.2 Configuration of 12.5 MG Pond at Graham's Pond Site

Ways to construct a pond at the Graham's Pond site with the design capacity of 12.5 MG were explored, using AutoCAD Civil 3D. A new pond would be designed using the same guidelines for side slopes and access as used for pond configurations considered for Alternative 3 (Section 4,

Design Criteria). Options included digging out within the existing pond footprint, expanding the existing footprint, and bringing the dam forward (northerly). Design would also have to include a by-pass for the creek water that currently flows into the pond.

It was determined that it is not possible to construct a 12.5 MG pond with a dam less than 25 feet high at the site of Graham's Pond, based on site topographic limitations. The most appropriate configuration, given the topographic restrictions, would require construction of a dam approximately 35 feet high, which would require DSOD permitting.

Serpentinite soils have been identified at the east abutment of the dam and along the southeasterly side of Graham's Pond. If construction of a pond at the site is carried forward for analysis as a Project alternative, laboratory testing of a rock/soil sample for asbestos would be included as part of the analysis.

### 6.1.3 Conflicts Associated with Construction of a New Pond at Graham's Pond Site

Construction of a lined, covered pond with a new dam at the Graham's Pond site would require the existing pond to be drained and kept out of service for the construction period. There are practical issues that would have to resolved, including storage of the OCSD effluent and by-pass of creek flow during construction, to allow the pond to be taken out of service. There may be wastewater-derived sediment on the floor of the pond, which would have to be evaluated and properly disposed.

Constraints on expansion of Graham's Pond include the existing steep slopes alongside the pond. The site would not accommodate practically a 12.5 MG pond with an open channel creek by-pass, and the creek would be routed through a five-foot diameter culvert alongside the pond. An inlet structure would be designed to route creek flows into the culvert. The creek bed adjoining the dam would require modification. Permitting for creek construction would be severely limiting, and it is likely that the creek by-pass would have to be completed in the construction season before the pond itself was reconstructed.

### 6.1.4 Water Rights

The land-owners possess two Water Rights for storage and diversion of water at Graham's Pond. If the pond were to be reconfigured as an off-stream effluent storage pond, the Loades would have no way to store the stream water. There would be some risk of losing the Water Rights because they were not being exercised.

### 6.1.5 Land-Owner Opposition

The land-owners have stated that they want Graham's Pond returned to their use. They have stated that they will oppose a project to construct an effluent storage pond at the Graham's Pond site, and that they will not necessarily oppose construction of a pond at another site on their lands.

### 6.1.6 Environmental Mitigation

Graham's Pond provides habitat for birds and animals. If the existing pond is replaced by a covered effluent storage pond, that habitat would be lost. Lost habitat would have to be replaced, at a ratio of a minimum of one-to-one, and as high as three-to-one, within the Dutch Bill Creek watershed. Securing a location for this mitigation would be very challenging, potentially requiring construction of a new on-stream dam and pond, or equivalent.

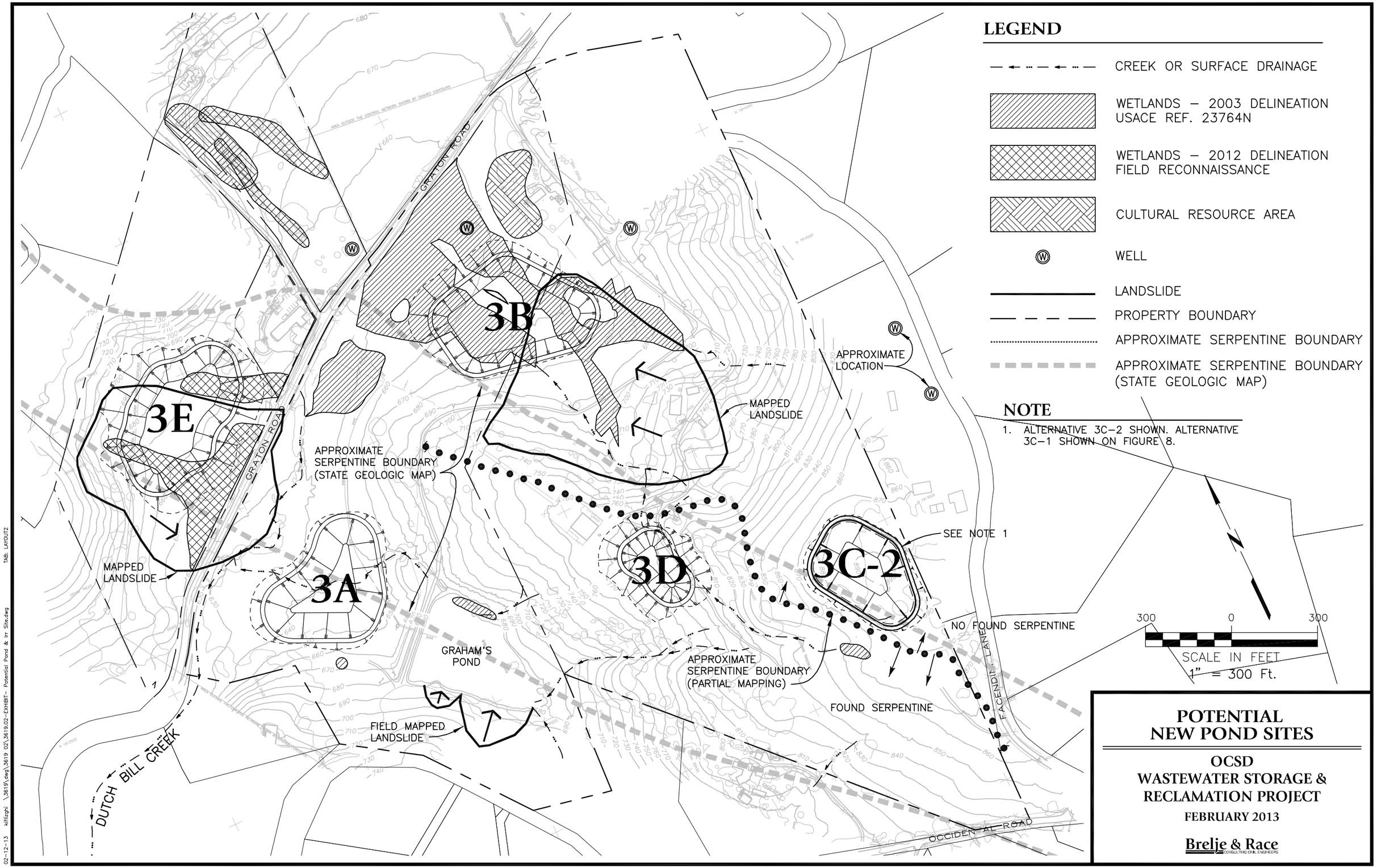



FIGURE 4

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### 6.1.7 Evaluation of Alternatives 1 and 2

Effluent Storage Pond Alternatives 1 and 2 do not meet Project objectives for a pond that provides the design capacity and keeps to a dam height less than 25 feet. Issues of constructibility, potential loss of the land-owners' existing Water Rights, and land-owner desire to regain use of Graham's Pond create significant barriers to development of Alternatives 1 and 2. In addition, mitigation costs associated with loss of pond habitat render these alternatives expensive and possibly infeasible.

## 6.2 ALTERNATIVE 3

The five potential pond configurations developed for Alternative 3 are shown in Figure 4, and discussed in order below.

### 6.2.1 Potential Pond Site 3A

Potential Pond Site 3A is located on the Loades parcel, northwesterly and down slope from Graham's Pond. Slopes in the area are gentle to moderate. The drainage from Graham's Pond flows through the area, and a creek flows just to the north. A configuration was developed for a 12.5 MG pond at the site, which would have a maximum dam height of less than 25 feet and which would require no import or export of materials. Maximum water depth would be 15 feet, with three feet of operating freeboard. Salient characteristics of construction at this site include the following:

- There is potential for liquefaction of the native alluvial materials, requiring deep excavation to obtain a good foundation.
- Geological maps indicate serpentinite upslope of and underlying part of the site. If construction of a pond at the site is carried forward for analysis as a Project alternative, laboratory testing of a rock/soil sample for asbestos would be included as part of the analysis.
- The site is very visible from Graton Road
- There are no known drinking water wells within 100 feet of the pond site.
- The stream that flows northerly from Graham's Pond would have to be re-aligned and mitigation made for the loss of the existing stream section. Riparian planting along the re-aligned creek and the existing Dutch Bill Creek could assist with mitigating visual impacts and balance the loss of creek habitat.
- Approximately 0.03 acre of wetlands was identified in 2003 within the site (Prunuske Chatham, Inc., wetland delineation, USACE Ref. 23764N). Any wetlands within the site would be lost and require mitigation.
- Approximately one acre of land currently irrigated with the District's recycled water would no longer be available for irrigation.
- The existing recycled water transfer main from the WTF would be extended approximately 800 feet from Occidental Road.
- The existing Graham's Pond irrigation pump station could be turned over to the land-owners for their use. The existing main from the pump station would be demolished during construction of Pond 3A.
- A new recycled water irrigation pump station, and a booster pump station for irrigating areas at higher elevation, would be constructed as part of a Pond 3A project.
- Irrigation mains would be constructed from the pond to the irrigation areas.

- Development of this pond alternative would include decommissioning use of Graham's Pond for effluent storage and return to land-owner for surface water storage.

### 6.2.2 Potential Pond Site 3B

Potential Pond Site 3B is located on the Loades parcel, northwesterly and down slope from the Loades' residence. Slopes in the area are gentle to moderate. A configuration has been developed for a 12.5 MG pond at the site which would have a maximum dam height of less than 25 feet. Maximum water depth would be 15 feet, with three feet of operating freeboard. Salient characteristics of construction at this site include the following:

- There is a historical or prehistorical landslide on approximately one third of the potential pond site.
- There is potential for liquefaction of the native alluvial materials.
- The site is very visible from Graton Road
- The site is near and in view of residences on the Loades property.
- Approximately 3 acres of wetlands were identified in 2003 within the site (Prunuske Chatham, Inc., wetland delineation, USACE Ref. 23764N). Any wetlands within the site would be lost and require mitigation.
- The site is near the land-owners' existing drinking water well. The potential pond has been configured to provide a 100-foot set-back as required under Title 22.
- Approximately 3 acres of land currently irrigated with the District's recycled water would no longer be available for irrigation.
- The existing transfer main would be extended approximately 1,800 feet from Occidental Road.
- The existing Graham's Pond irrigation pump station could be turned over to the land-owners for their use.
- A new recycled water irrigation pump station would be constructed as part of a Pond 3B project.
- Development of this pond alternative would include decommissioning use of Graham's Pond for effluent storage and return to land-owner for surface water storage.

### 6.2.3 Potential Pond Site 3C

Potential Pond Site 3C is located on the Loades parcel, at the southeast corner. The area is at the highest elevation on the property. Two configurations have been developed for 12.5 MG ponds at the site, both of which would have a maximum am height of less than 25 feet. Maximum water depth would be 37 feet, with three feet of operating freeboard. Salient characteristics of construction at this site include the following:

#### 6.2.3.1 Site 3C-1

- The site is underlain by rock, approximately equal parts serpentinite mélange and sandstone, shale, and conglomerate. Initial near-surface geotechnical investigation indicates that the pond could be excavated using standard construction equipment.
- If construction of a pond at the site is carried forward for analysis as a Project alternative, laboratory testing of rock/soil samples for asbestos would be included as part of the analysis.

- The potential pond has been configured as a dug-in reservoir, so that it would not obstruct existing westerly views from the neighboring parcels to the east.
- Although the initial design criteria called for side slopes of 3:1, soils encountered in test pits appear capable of construction at steeper slopes. The pond has been configured with inside slopes of 2:1 and outside slopes of 3:1 in order to reduce the overall footprint.
- Approximately 100,000 cubic yards (CY) of earth and rock would be excavated to construct the pond. This material would have to be hauled off the site. Approximately half the material would be serpentinite.
- A home is located approximately 300 feet easterly of the site. Although there is no record of a well, it is believed that a drinking water well was constructed approximately 130 feet from the parcel line.
- Portions of the pond would be visible from Occidental Road looking easterly to northeasterly.
- Approximately 0.1 acre of wetlands was identified in 2003 within the site (Prunuske Chatham, Inc., wetland delineation, USACE Ref. 23764N). Any wetlands within the site would be lost and require mitigation.
- The land-owner has formed a preliminary agreement for location of a cell tower within the proposed pond site. A pond project at the site would need to include relocation of the cell tower.
- No land currently irrigated with the District's recycled water would be affected.
- The existing transfer main would be extended approximately 3,600 feet from Graham's Pond.
- Because the pond site is approximately 100 feet higher in elevation that the WTF, transfer pumping would be provided at the WTF.
- The existing recycled water irrigation pump station could be transferred to the land-owners for irrigation from Graham's Pond under their water rights.
- Irrigation mains would be constructed from the pond to the irrigation areas.
- An irrigation pump station would not be required for irrigation areas that are at lower elevations. A small irrigation pump station would be needed for areas adjoining the pond.
- Development of this pond alternative would include decommissioning use of Graham's Pond for effluent storage and return to land-owner for surface water storage.

#### 6.2.3.2 Site 3C-2

- The site has been configured to minimize disturbance of serpentinite rock, thereby avoiding the potential issue of disposal of asbestos-containing materials.
- The site is underlain by rock, primarily sandstone, shale and conglomerate and keratophyre and quartz keratophyre tuff. Geotechnical investigation indicates that the pond could be excavated using standard construction equipment.
- Although the initial design criteria called for side slopes of 3:1, soils encountered in test pits appear capable of construction at steeper slopes. The pond has been configured with inside slopes of 2:1 and outside slopes of 2.5:1 in order to reduce the overall footprint.
- The potential pond has been configured as a dug-in reservoir, in order to minimize visual impacts from the neighboring parcels to the east.

- Approximately 92,600 cubic yards (CY) of earth and rock would be excavated to construct the pond. Approximately 90% of this material would have to be hauled off the site. Geotechnical evaluation indicates that the material would be usable as a construction material.
- Approximately 1.7 acres of trees would be removed for construction at this site.
- A home is located approximately 200 feet to the east of the site. Although there is no record of a well, it is believed that a drinking water well was constructed approximately 130 feet from the parcel line.
- Portions of the pond would be visible from Occidental Road looking easterly to northeasterly, although less than Alternative 3C-1.
- It appears that no wetlands are within the proposed pond footprint.
- No land currently irrigated with the District's recycled water would be affected.
- The existing transfer main would be extended approximately 3,800 feet from Occidental Road.
- Because the pond site is approximately 100 feet higher in elevation that the WTF, transfer pumping would be provided at the WTF.
- Irrigation mains would be constructed from the pond to the irrigation areas.
- An irrigation pump station would not be required for irrigation areas that are at lower elevations. A small irrigation pump station would be needed for areas adjoining the pond.
- Development of this pond alternative would include decommissioning use of Graham's Pond for effluent storage and return to land-owner for surface water storage.
- The existing recycled water irrigation pump station could be transferred to the land-owners for irrigation from Graham's Pond under their water rights.

### 6.2.4 Potential Pond Site 3D

Potential Pond Site 3D is located on the Loades parcel, southwesterly of the Loades residence and easterly of Graham's Pond. It was not possible to develop a pond configuration for a 12.5 MG pond at the site. The largest pond that could be practicably constructed would have a capacity of approximately 1.9 MG. Maximum water depth would be 15 feet, with three feet of operating freeboard. Salient characteristics of construction at this site include the following:

- It was not possible to develop a pond configuration for a 12.5 MG pond at the site.
- There would be no net import or export of earth needed.
- There are no known drinking water wells in the vicinity of the site.
- The existing transfer main would be extended approximately 1,500 feet from the location where it currently goes into Graham's Pond.

### 6.2.5 Potential Pond Site 3E

Potential Pond Site 3E is located on the Colella parcel, across Graton Road from Graham's Pond. A configuration has been developed for a 12.5 MG pond at the site which would have a maximum dam height of less than 25 feet. Maximum water depth would be 15 feet, with three feet of operating freeboard. Salient characteristics of construction at this site include the following:

- There is a historical or prehistorical landslide on approximately two thirds of the potential pond site.
- The pond dike would be highly visible from Graton Road. The top of dike would be approximately 30 feet higher than the adjoining section of Graton Road and present a length of approximately 600 feet along the road.
- Geological maps indicate serpentinite upslope of and underlying part of the site. If construction of a pond at the site is carried forward for analysis as a Project alternative, laboratory testing of a rock/soil sample for asbestos would be included as part of the analysis.
- The nearest known well is on the same parcel as the pond site, approximately 450 feet away and up-gradient.
- Preliminary assessment indicates there may be two or more acres of wetlands and approximately 900 feet of perennial stream in the site. These areas would be lost during pond construction and require mitigation.
- No land currently irrigated with the District's recycled water would be affected.
- The existing recycled water transfer main would be extended approximately 1,800 feet from the location where it currently enters Graham's Pond.
- Transfer and irrigation mains would be constructed beneath Graton Road.
- The existing Graham's Pond irrigation pump station could be turned over to the land-owners for their use.
- A new recycled water irrigation pump station would be constructed as part of a Pond 3E project.
- Development of this pond alternative would include decommissioning use of Graham's Pond for effluent storage and return to land-owner for surface water storage.

### 6.2.6 Evaluation of Potential Alternative 3 Pond Sites

The five sites were evaluated for ability to meet the Project objective of identifying a site where a pond could be constructed that met the design criteria. In consultation with the District, potential Sites 3B, 3D, and 3E were eliminated from consideration. Potential site 3D was eliminated because it was not possible to construct a large enough pond, even for use as the auxiliary pond for Alternative 1. Potential sites 3B and 3E were removed from consideration, due to construction issues associated with landslides over the sites, as well as the other considerations listed. Potential Sites 3A and 3C were both viewed as more constructible than other alternatives in spite of uncertainties about geotechnical conditions. Alluvial materials at Site 3A may be subject to liquefaction, requiring deep excavation to obtain a good foundation. Site 3C is underlain by rock, including serpentinite mélange. In order to construct a pond with adequate capacity on Site 3C, excavation would be deep, and in the range of 85,000-100,000 cubic yards of excess material would be produced. Site 3C-1 was deemed unsuitable because of the large proportion of serpentinite within the pond footprint.

## 6.3 SELECTION OF POND SITES FOR FURTHER ANALYSIS

Based upon the analysis above, potential pond Site 3A and Site 3C-2 were carried forward as Project alternatives for further analysis. Site 2 was also carried forward, even though it did not meet the

project objective for maximum dam height, to retain the possibility of continued use of the Graham's Pond site. Section 7 presents a discussion of these project alternatives.

## 6.4 IRRIGATION AREAS

Design criteria include development of 24 acres of pasture or feed crop irrigation area to meet long-term needs. At the projected ADWF, an area of 24 acres would be needed to assure capacity to empty the storage pond after the 100-year peak rainfall year. Irrigation development would include construction of pump stations and transmission piping. The potential irrigation areas would be much the same for any pond site, except for the land needed for development of the pond site itself.

The existing irrigation areas consist of approximately 12 acres on the northeast portion of the Loades property, plus a small area adjoining the pump station near the Graham's Pond dam. Potential additional irrigation areas were delineated within the restrictions discussed in Section 4, specifically avoidance of areas slopes greater than 15%, with maximum slopes of 20%, and providing a set-back from wetlands and natural watercourses. Figure 5 shows existing and potential irrigation areas. The irrigation areas are summarized in Table 4.

**Table 4: Existing & Potential Irrigation Areas**

| Area | Area, acres | Land-Owner | Current Land Use | Usable Acres for Pond Sites | | |
|---|---|---|---|---|---|---|
| | | | | 2 | 3A | 3C-2 |
| E-1 | 0.3 | Loades | Pasture | 0.3 | 0 | 0.3 |
| E-2 | 12 | Loades | Pasture | 12 | 12 | 12 |
| P-1 | 0.4 | Loades | Pasture | 0.4 | 0.3 | 0.4 |
| P-2 | 1.9 | Loades | Pasture | 1.9 | 0 | 1.9 |
| P-3 | 2.5 | Loades | Pasture | 2.5 | 0 | 2.5 |
| P-4 | 1.1 | Colella | Open Land | 1.1 | 1.1 | 1.1 |
| P-5 | 0.7 | Briggs | Pasture | 0.7 | 0.7 | 0.7 |
| P-6 | 0.9 | Loades | Pasture | 0.9 | 0.9 | 0.9 |
| P-7 | 6.8 | Loades | Pasture | 6.8 | 6.8 | 4.7 |
| P-8 | 1.5 | Loades | Pasture | 1.5 | 1.5 | 1.5 |
| **Total Existing and Potential Irrigation Area Associated with Each Pond Site, acres** | | | | **28.1** | **23.3** | **26.0** |

As can be seen, it appears possible to develop the required 24 acres irrigation area within the study area, for any of the pond sites selected for further consideration except Site 3A. Construction of a pond at Site 3A would displace approximately 4.8 acres of existing and potential irrigation area. It appears that, for Site 2 and Site 3C-2, it would be possible to achieve the required 24 acres on the Loades parcel alone.

LEGEND
CREEK OR SURFACE DRAINAGE
WETLANDS – OR RIPARIAN AREA
EXISTING IRRIGATION AREA
E–1
POTENTIAL IRRIGATION AREA
P–1
WELL
RECORDED WELL *
* WHERE RECORDS DON'T INDICATE WELL LOCATION, WELL IS SHOWN AT CENTER OF PARCEL.
300 0 300
SCALE IN FEET
1" = 300 Ft.
EXISTING & POTENTIAL IRRIGATION AREAS
OCSD
WASTEWATER STORAGE & RECLAMATION PROJECT
FEBRUARY 2013
Brelje & Race
CONSULTING CIVIL ENGINEERS
GRATON ROAD
GRATON ROAD
DUTCH BILL CREEK
OCCIDENTAL ROAD
FACENDINI LANE
APPROXIMATE LOCATION
P–1
P–2
P–3
P–4
P–5
P–6
P–7
P–8
E–1
E–2
WR
W
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FIGURE 5

# 7. DESCRIPTION OF PROJECT ALTERNATIVES

## 7.1 GENERAL

The previous section presented site evaluations for all project options. Several sites were eliminated from further evaluation because of high risks associated with construction, regulatory restrictions, geotechnical challenges and anticipated high costs. Three alternatives were identified for further consideration. These alternatives include:

Alternative 2: Construction of 12.5 MG reservoir in same location as existing reservoir (Graham's Pond), with new DSOD dam, stream by-pass, liner and cover, and establishment of 24 acres of pasture irrigation, with appurtenant pumping and transmission facilities.

Alternative 3A: Construction of 12.5 MG reservoir with liner and cover, northwesterly and downslope of the existing reservoir, at Site 3A, and establishment of 24 acres of pasture irrigation, with appurtenant pumping and transmission facilities.

Alternative 3C-2: Construction of a 12.5 MG reservoir with liner and cover, on the southeast portion of the Loades property, at Site 3C and outside the area of serpentinite mélange, and establishment of 24 acres of pasture irrigation, with appurtenant pumping and transmission facilities.

This section presents a summary of the preliminary geotechnical analysis and cost estimates for these project alternatives. Information about the particular sites that was presented in Section 6 is generally not repeated in this section.

### 7.1.1 Geotechnical Study

Design level geotechnical investigations were conducted for sites 3A and 3C-2.

### 7.1.2 Cost Estimates

A cost estimate was developed for each of the three project alternatives, based upon the preliminary designs described herein. Construction cost estimates include earthwork, drainage, pond liner and cover, development of vegetated swale(s) for water pumped off the pond covers, pumping, piping, irrigation infrastructure, electrical, access roads, disposal of excess excavated materials, surface restoration, fencing around each pond, and permitting. Construction cost estimates were based upon typical current costs for construction projects bid under public contract code.

The cost estimates present a range of potential costs associated with disposition of excess earth materials. Two of the pond alternatives would include export of serpentine materials. No local quarry was found that would accept these materials for construction use. The nearest Class III landfill did not provide a clear indication of potential tipping fees for the serpentinite materials. Two of the alternatives involve export of sandstone, conglomerate, tuff, shale and graywacke materials. Geotechnical evaluation has indicated that these materials are suitable for construction use as engineered fill. The cost for export would vary depending upon market conditions at the time the project is constructed, as well as upon the quality of the materials. The range of costs presented for earthwork represent the likely range of low and high costs for the disposition of excess materials.

The estimate of capital costs included the estimated costs for required environmental mitigations, engineering design and construction management at 25% of construction, environmental documents, and administrative and legal costs. A contingency of 15% was added into the estimate of total capital costs for each project alternative.

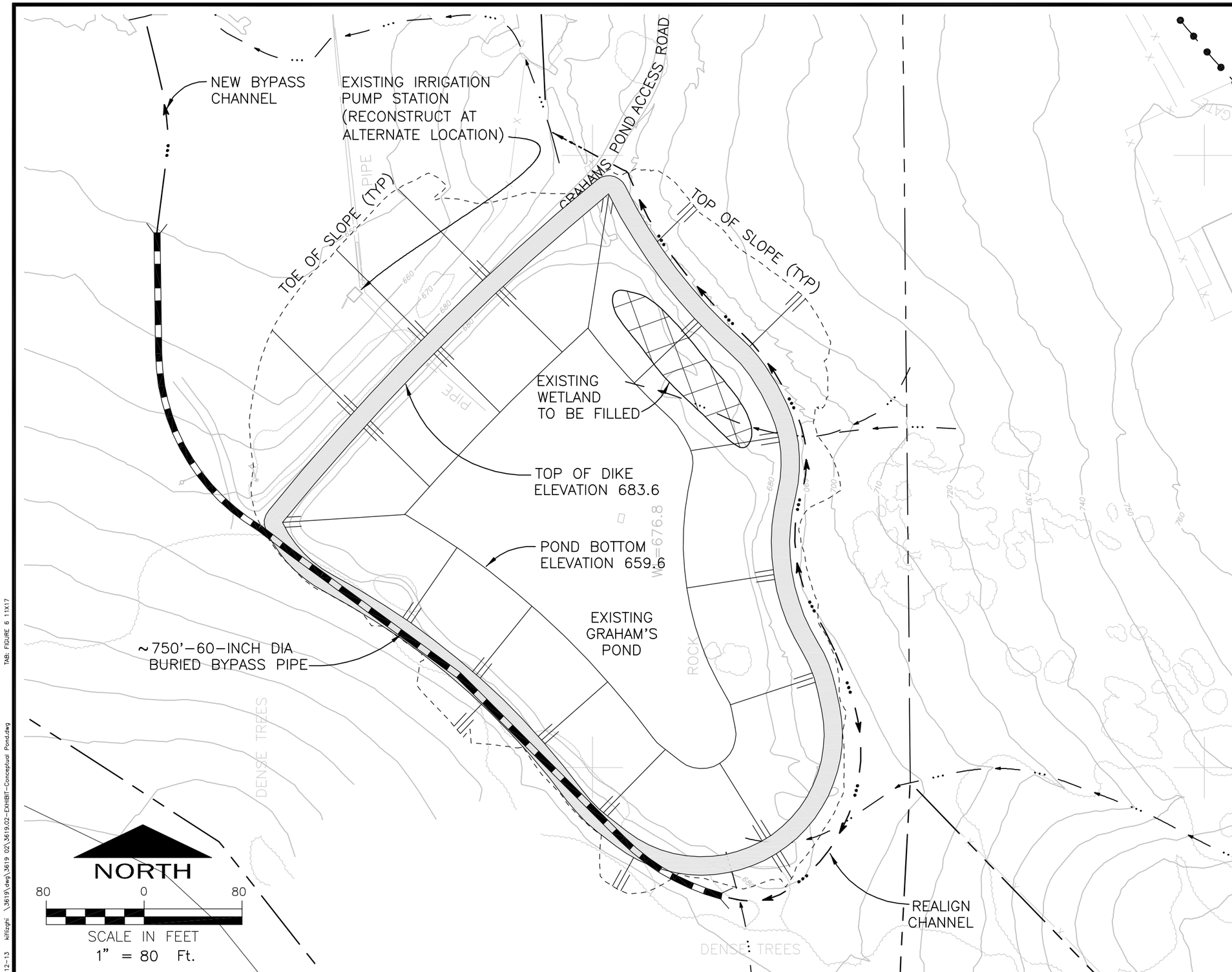
Cost estimates presented herein do not include potential costs for easements, lease or purchase of areas for irrigation and/or pond construction. These costs would be negotiated between the District and the landowner, based upon appraisals of property values. Negotiations with the land-owner and determination of these costs would be undertaken in a subsequent phase of the project.

### 7.1.3 Restoration of Graham's Pond

Development of any of the pond alternatives would potentially include characterization, removal and disposal of wastewater sediments from Graham's Pond. Characterization of the sediments was outside the scope of the 30% conceptual design, and costs would be highly dependent upon the volume and chemical character of the sediments. Removal of wastewater sediments would be a part of construction of Pond Alternative 2. If the District constructs a pond at a new location, the WDRs for the new storage and irrigation system can be expected to include a requirement that Graham's Pond be decommissioned to a condition such that it was not a threat to water quality (Appendix B). Additionally, the current Agreement between the District and the landowners includes terms for return of the pond to the landowners, including water quality testing and pond restoration if it is needed. Because of the high level of uncertainty regarding restoration requirements, and because costs would be similar for any of the alternatives, cost estimates did not include potential costs associated with removing wastewater sediments from, or otherwise restoring, Graham's Pond.

## 7.2 ALTERNATIVE 2

Development of Pond Alternative 2 would include removal of the existing Graham's Pond dam, excavation of rock and soils from the bottom and sides of Graham's Pond, and construction of a new 35-foot high dam. General configuration, liner and cover and other features would be as set forth in Section 4, Design Criteria. Figure 6 shows the conceptual plan of Pond Alternative 2.




## POND DATA ALT. - 2

VOLUME : 12.5 MILLION GALLONS

DEPTH : 24 FEET

MAX DIKE HEIGHT : 34 FEET

SIDE SLOPES
- INTERIOR: 3:1
- EXTERIOR FILL: 3:1
- EXTERIOR CUT: 2:1

DIKE ROAD WIDTH : 15 FEET

OPERATING FREEBOARD : 3 FEET

APPROXIMATE EARTHWORK – CUBIC YARDS
- EXCAVATION 33,000 CY
- EMBANKMENT 28,000 CY
- EXPORT 5,000 CY

FLOATING COVER : HYPALON

LINER : HDPE MEMBRANE

DSOD : JURISDICTIONAL

**POND ALTERNATIVE 2**
**NEW POND EXISTING GRAHAM'S SITE**
**CONCEPTUAL DESIGN**

**OCSD**
**WASTEWATER STORAGE & RECLAMATION PROJECT**
**FEBRUARY 2013**

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02-12-13 kifizghi \3619\dwg\3619 02\3619.02-EXHIBIT-Conceptual Pond.dwg TAB: FIGURE 6 11X17

**FIGURE 6**

### 7.2.1 Geotechnical Findings

The northeasterly half of this pond site is underlain by serpentinite mélange. Serpentinite outcrops are visible along the eastern side of Graham's Pond. A sample of discrete small rocks and rock fragments was taken from the serpentinite on the east side of Grahams Pond, approximately 50 foot south of the overflow culvert, and analyzed for asbestos content. Asbestos was found in the sample at a concentration of 2.5% (Laboratory report for asbestos testing is in Appendix E.). The asbestos in the single sample that was tested would be classified as non-friable, as the sample consisted of rock fragments. Further sampling and testing would be required before design and construction of a pond at this site. Friable asbestos may be found in soils associated with the rock outcropping and in sediments on the bottom of the pond. Any materials with friable asbestos at concentrations greater than 1% would be subject to hazardous waste disposal regulations if removed from the site.

Because of the presence of naturally occurring asbestos (NOA) at the Alternative 2 site, construction at the site would be subject to California Air Resources Board (CARB) regulation, as described in Technical Memorandum No. 2, Appendix B. Construction would require development and implementation of an Asbestos Dust Mitigation Plan in accordance with CARB regulations.

### 7.2.2 Preliminary Estimate of Capital Costs

A preliminary estimate of capital costs associated with construction of Pond Alternative 2 is summarized in

Table 5. A significant feature of the costs is the estimated cost of mitigating for the loss of pond habitat that would result from converting Graham's Pond into a covered effluent storage pond. The dam construction and design costs would be increased by approximately 20% because of the requirements of constructing a DSOD dam. The range of costs presented for earthwork represent the likely range of low and high costs for the disposition of excess materials, including disposal of asbestos-bearing materials. Liner costs reflect installation of a membrane liner with subdrain system.

**Table 5: Preliminary Estimate of Probable Costs for Pond Alternative 2**

| Project Element | Estimated Cost, million dollars |
|---|---|
| Construction: | |
| Pond construction | $ 0.56 |
| Export of earth materials | **$ 0.43 - $ 0.74** |
| Surface drainage | $ 0.34 |
| Liner and cover, subdrain system | $ 0.88 |
| Misc (fencing, roads, erosion control) | $ 0.18 |
| Transfer pump station & main | $ - |
| Irrigation System | $ 0.59 |
| **Estimated Total Construction Costs** | **$ 3.0 - $ 3.3** |
| Engineering Design & Construction Management | $ 0.69 |
| Administrative & Legal | $ 0.15 |
| Environmental Document | $ 0.26 |
| Permitting Applications | $ 0.11 |
| Environmental Mitigation | $ 2.05 |
| 15% Contingency | $ 0.94 |
| **Estimated Total Capital Costs** | **$ 7.2 - $ 7.6** |

*Miscellaneous costs include fencing, access roads, erosion control and construction of a vegetated swale.
Cost estimate does not include potential restoration of Graham's Pond or costs for easement, lease or purchase of site for pond construction.

## 7.3 ALTERNATIVE 3A

Initial preliminary design layouts for site 3A were based on the basic design criteria established in Section 4. Overall pond dike heights were held to a maximum of 25 feet, depth of water was approximately 15 feet, and side slopes were 3:1 for interior and exterior dikes. A 30-foot setback was held from Dutch Bill Creek to the toe of the pond dike to allow space for planting trees to screen the view from Graton Road and improve the creek habitat. The elevations for the top of dike and pond bottom were set so that the desired volume of 12.5 million gallons was achieved and the earthwork balanced (meaning all excavated materials would be used in the construction of the embankments with no off haul from the site). This pond configuration is shown in Appendix D, Figure 1.

### 7.3.1 Geotechnical Findings and Design Implications

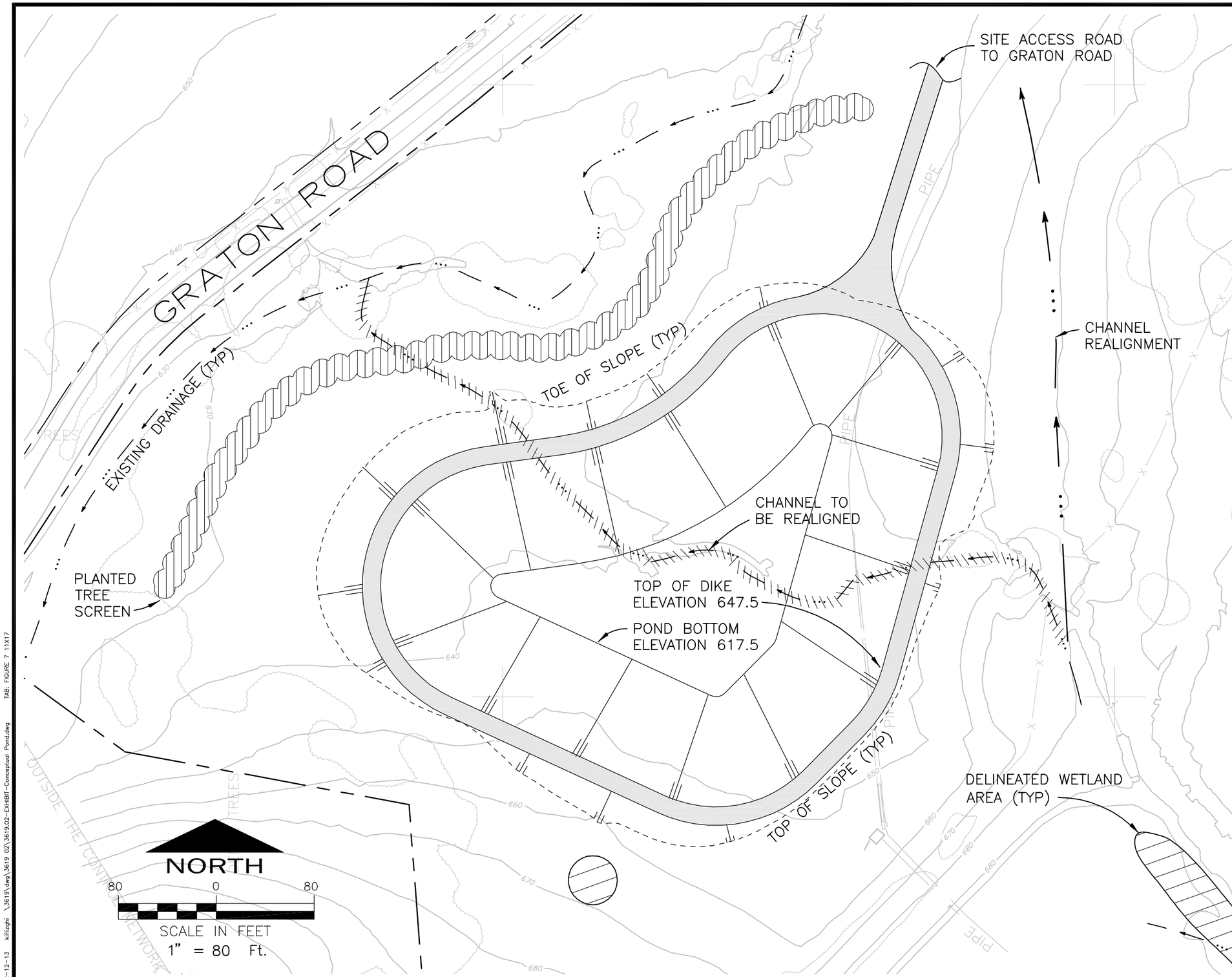
After the site was selected as a Project Alternative, six cone penetrometer tests (CPT Soundings) were performed, followed by four exploratory geotechnical borings within the proposed footprint (Geotechnical Report, Appendix C) to assess the site subsurface conditions.

The subsurface investigation revealed that the materials underlying the site consisted of approximately 10 to 15 feet thick Quaternary alluvial deposits which overlie serpentine formation bedrock. The alluvial deposits are composed of layered silty clays to clayey sands and are generally very soft to soft in material consistency. The clayey sand layers are generally very loose to loose, contain high amount of fines, and are of relatively low shear strength. The thickness of the alluvial deposits generally increases towards the north-west side of the site. Atterberg limits and grain size analysis testing performed on these materials show these materials will behave more like clay and are generally resistant to significant pore pressure increases and associated loss of shear strength

(liquefaction) during earthquakes. The underlying serpentine formation bedrock is very closely fractured and sheared, friable and behaves more like a very stiff silty/sandy clay. Although the serpentine is highly variable, it is more favorable as a foundation for embankments than are the alluvial deposits.

The clay rich alluvial deposits would be very challenging to use as a material for dike construction, due to high plasticity, non-uniform and potentially low shear strength, poor workability and ability to mix with other materials, high expansion potential, and tendency to hold water. Groundwater was found near the ground surface; the high groundwater, which is in part due to poor drainage in the clay soil, creates additional challenges during construction as well as in long term sub-surface drainage management. Preliminary analysis based on simplified assumptions of parameters revealed that there would likely be differential settlement (caused by proposed embankment loading of the alluvial deposits) across the pond section, with up to two feet settlement in places over the long term. Seismic loading calculations revealed that a factor a safety of only 1.2 would be achieved, where a minimum factor of safety for design purposes would be 1.5. Additives to stabilize the clay material and make it into a suitable material for dike construction were considered and dismissed as too expensive. An option for construction was considered that would include leaving the clay generally in place, importing material and building the dikes on top of the clay. Geotechnical analysis revealed that this approach would be unsatisfactory due to the low strength characteristics of the clay (alluvial deposits) creating the potential for significant differential settlement. It was determined that the most feasible approach to the poor foundation materials was to remove them.

A different general pond configuration was then developed, which would involve removing the clay from the entire footprint of the pond to a depth of approximately eight to twelve feet, and




## POND DATA ALT. - 3A

VOLUME : 12.5 MILLION GALLONS

DEPTH : 30 FEET

MAX DIKE HEIGHT : 15 FEET

SIDE SLOPES
INTERIOR: 3:1
EXTERIOR: 3:1

DIKE ROAD WIDTH : 15 FEET

OPERATING FREEBOARD : 3 FEET

APPROXIMATE EARTHWORK – CUBIC YARDS
EXCAVATION 85,000 CY
SUBEXCAVATION 13,000 CY
EMBANKMENT 26,000 CY
EXPORT 59,000 CY

FLOATING COVER : HYPALON

LINER : HDPE MEMBRANE

DSOD : NON–JURISDICTIONAL

## POND ALTERNATIVE 3A CONCEPTUAL DESIGN

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FIGURE 7

constructing a lower profile pond. Figure 7 shows the reconfigured conceptual plan of Pond Alternative 3A. This new pond configuration would require a greater water depth to achieve the required 12.5 million gallon volume. Groundwater would be encountered during construction, and excavations would require dewatering. Excavation with ongoing dewatering introduces elevated risks and increases construction costs. Design and construction would include permanent sub-drains around the entire pond. Approximately 60,000 CY of clayey materials would have to be off-hauled under this alternative. Because the clay has poor strength characteristics, local quarries would not want it, and an alternate disposition would need to be secured.

A sample of the soil and alluvial deposits was taken from the eroded cut of the stream meander below Graham's Pond, at 5 feet deep, on the east side of the creek bank, approximately 75 feet south of Boring No. B4, and analyzed for asbestos content. The sample consisted of soil containing pebbles up to approximately 0.5-inch diameter. Laboratory analysis of the sample found an asbestos concentration of 0.75% (The laboratory report for asbestos testing may be found in Appendix E.). Further sampling of the alluvial materials and testing for asbestos would be required before design and construction of a pond at this site. Any materials with friable asbestos at concentrations greater than 1% would be subject to hazardous waste disposal regulations if removed from the site.

The geotechnical investigation revealed that serpentine rock exists at variable depths beneath the clay soils at the Pond 3A site. The subsurface serpentine bedrock was highly variable, generally consisting of highly to completely weathered, friable to plastic strata often with a soil-like consistency and associated engineering properties, with entrained less weathered and more competent cobbles, blocks and lenses, which is typical of the serpentine matrix mélange formation. Excavation of some of this serpentine would be included in the construction of a pond at this site. Final design of a pond would need to address the quantity of serpentine and whether it could be used in on-site construction or would have to be off-hauled.

Because of the presence of NOA at the Alternative 3A site, construction at the site would be subject to CARB regulation. Construction of Pond Alternative 3A would require development and implementation of an Asbestos Dust Mitigation Plan in accordance with CARB regulations. If the fatty clay is demonstrated to contain asbestos, disposition of the material would be further complicated beyond the issues of poor strength and high expansiveness.

If this site is selected as the preferred alternative, a more significant site geotechnical investigation and laboratory testing program would be needed. Design would include detailed directions for excavation, materials separation, and use of particular materials in different parts of the pond construction.

### 7.3.2 Preliminary Estimate of Capital Costs

A preliminary estimate of capital costs associated with construction of Pond Alternative 3A is summarized in Table 6. A significant feature of the estimated costs is the special construction procedures necessary due to the nature of the clay at the site. The required off haul of the clay material is a costly component of the development of site 3A. The range of costs presented for earthwork represent the likely range of low and high costs for the disposition of excess materials. Liner costs reflect installation of a membrane liner with subdrain system.

**Table 6: Preliminary Estimate of Probable Costs for Pond Alternative 3A**

| Project Element | Estimated Cost, million dollars |
|---|---|
| Construction: | |
| Pond construction | $ 0.89 |
| Export of earthen materials | **$ 4.3 - $ 7.4** |
| Surface drainage | $ 0.21 |
| Liner and cover, subdrain system | $ 0.81 |
| Miscellaneous* | $ 0.16 |
| Transfer Pump and piping | $ 0.05 |
| Irrigation System | $ 0.59 |
| **Estimated Total Construction Costs** | **$ 7.1 - $ 10.2** |
| Engineering Design & Construction Management | $ 1.17 |
| Administrative & Legal | $ 0.15 |
| Environmental Document | $ 0.26 |
| Permitting Applications | $ 0.11 |
| Environmental Mitigation | $ 0.31 |
| 15% Contingency | $ 1.36 |
| **Estimated Total Capital Costs** | **$ 10.4 - $ 14.4** |

*Miscellaneous costs include fencing, access roads, erosion control and construction of a vegetated swale.
Cost estimate does not include potential restoration of Graham's Pond or costs for easement, lease or purchase of site for pond construction.

## 7.4 ALTERNATIVE 3C-2

Figure 8 shows the conceptual plan of Pond Alternative 3C-2. This pond alternative was developed to evaluate the possibility of constructing a pond that would be entirely outside the area of serpentinite mélange. In the design layout for site 3C-2, the outside pond dike height was held to a maximum of 10 feet, in order to reduce the site footprint and minimize tree loss, the depth of water would be approximately 37 feet, and side slopes would be 2.5:1 for exterior dikes and 2:1 inside the pond. A 10 foot setback was held from the adjoining property line. The elevation for the top of dike was set so that the dike would not obstruct the existing line of sight from the adjoining parcel on the east. The desired volume of 12.5 million gallons was achieved with an excess of earth of approximately 83,650 CY to be hauled off site.

The proposed Pond Alternative 3C-2 site currently includes approximately 1.7 acres of mature mixed evergreen forest.

### 7.4.1 Geotechnical Findings

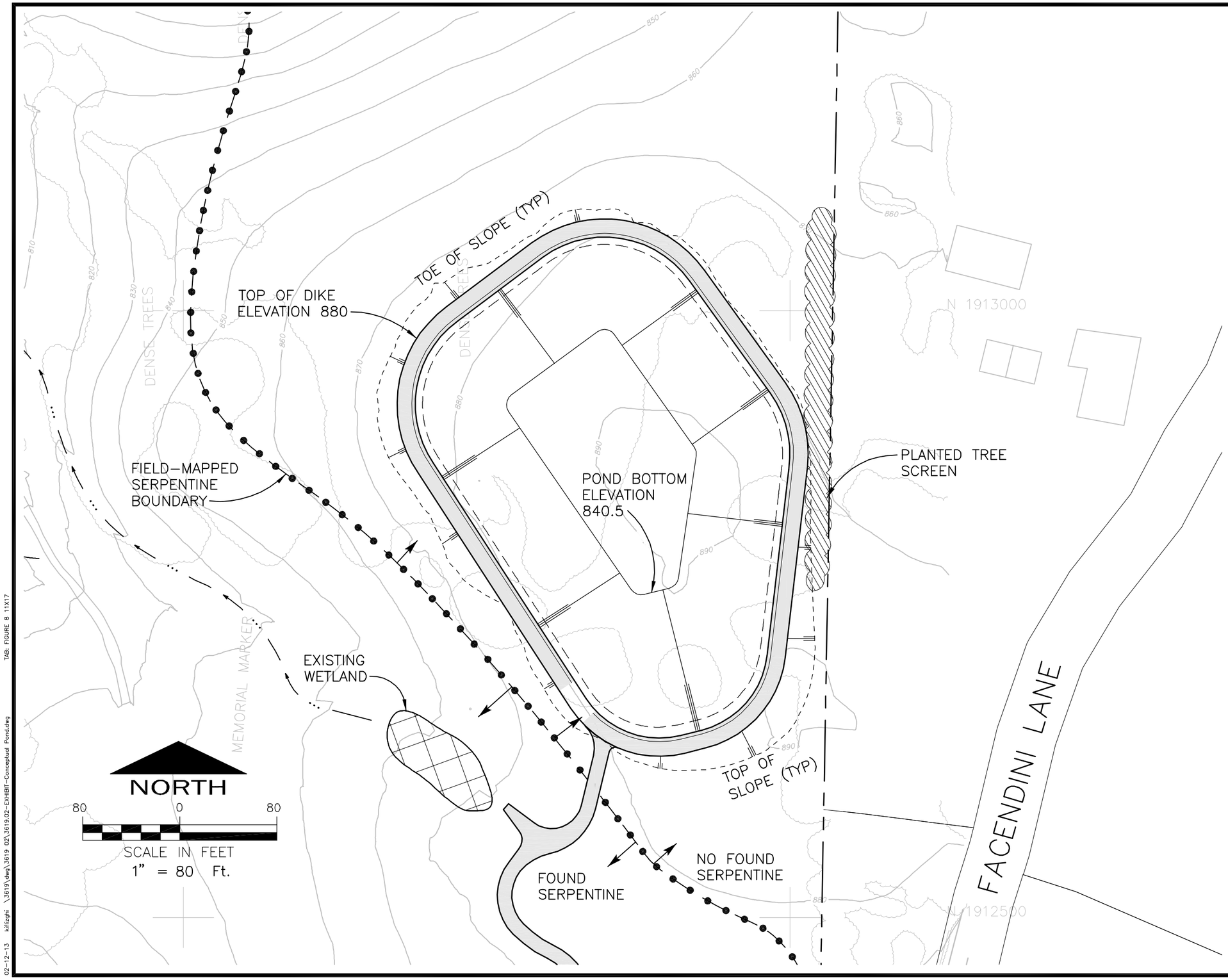
The Alternative 3C-2 site is underlain by sandstone, shale, conglomerate and graywacke. Findings are discussed in further detail in the Geotechnical Report, Appendix C.

A random mix sample of soil, small rock and rock fragments was taken from several of the test pits, and analyzed for asbestos content. The results are summarized in Table 7. A copy of the laboratory report may be found in Appendix E.

**Table 7: Asbestos Concentrations in Pond 3C Test Pit Samples**

| Test Pit | Native Material | Asbestos Concentration |
|---|---|---|
| 1 | Serpentinite Mélange | 2.3% |
| 2 | Serpentinite Mélange | 0.75% |
| 3 | Serpentinite Mélange | 0.25% |
| 4 | Sandstone, Shale and Conglomerate | ND |
| 5 | Sandstone, Shale and Conglomerate | ND |
| 6 | Serpentinite Mélange | 2.3% |

All four samples taken from serpentinite mélange were found to have asbestos at levels requiring construction practices in compliance with CARB regulations. Two of the samples had asbestos at concentrations above the hazardous waste limit for friable asbestos. The pond design has been configured to avoid excavation of serpentinite, in order to avoid issues associated with disposal of asbestos-bearing materials.




## POND DATA ALT. - 3C-2

VOLUME : 12.5 MILLION GALLONS

DEPTH : 40 FEET

MAX DIKE HEIGHT : 10 FEET

SIDE SLOPES
INTERIOR: 2:1
EXTERIOR: 2.5:1

DIKE ROAD WIDTH : 15 FEET

OPERATING FREEBOARD : 3 FEET

APPROXIMATE EARTHWORK – CUBIC YARDS
EXCAVATION 92,600 CY
EMBANKMENT 1,050 CY
ON–SITE USE 7,900 CY
EXPORT 83,650 CY

FLOATING COVER : HYPALON

LINER : HDPE MEMBRANE

DSOD : NON–JURISDICTIONAL

## POND ALTERNATIVE 3C-2 CONCEPTUAL DESIGN

**OCSD**
**WASTEWATER STORAGE & RECLAMATION PROJECT**
**FEBRUARY 2013**

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**FIGURE 8**

The laboratory analysis of samples from Test Pits 4 and 5, indicate no detectable asbestos. A copy of the laboratory report may be found in Appendix E.

### 7.4.2 Preliminary Estimate of Capital Costs

A preliminary estimate of capital costs associated with construction of Pond Alternative 3C-2 is summarized in Table 8. For cost-estimating, it was assumed that the excavated materials would not include serpentinite. The range of costs presented for earthwork represent the likely range of low and high costs for the disposition of excess, non-serpentinite, materials. Liner costs reflect installation of a synthetic membrane liner with subdrain system.

**Table 8: Preliminary Estimate of Probable Costs for Pond Alternative 3C-2**

| Project Element | Estimated Cost, million dollars |
|---|---|
| Construction: | |
| Pond construction | $ 0.71 |
| Export of earthen materials | **$ 0.93 - $ 1.1** |
| Surface drainage | $ 0.03 |
| Liner and cover, subdrain system | $ 0.60 |
| Miscellaneous* | $ 0.12 |
| Transfer Pump and piping | $ 0.34 |
| Irrigation System | $ 0.43 |
| **Estimated Total Construction Costs** | **$ 3.2 - $ 3.3** |
| Engineering Design & Construction Management | $ 0.67 |
| Administrative & Legal | $ 0.15 |
| Environmental Document | $ 0.26 |
| Permitting Applications | $ 0.11 |
| Environmental Mitigation | $ - |
| 15% Contingency | $ 0.67 |
| **Estimated Total Capital Costs** | **$ 5.0 - $ 5.2** |

*Miscellaneous costs include fencing, access roads, erosion control and construction of a vegetated swale.
Cost estimate does not include potential restoration of Graham's Pond or costs for easement, lease or purchase of site for pond construction.

## 7.5 COMPARISON OF POND ALTERNATIVES

This section has presented geotechnical and cost information that, in combination with the information presented in Section 6, strongly influences the practicality of the Project Alternatives. Table 9 presents a summary matrix of the alternatives.

The Design Report has revealed several geological features within the project area which have limited the selection of sites for construction of an effluent storage reservoir. The geological features include several historical landslides, a band of serpentinite running through the project area, and fatty clay alluvial soils. Testing of soil and rock samples from the sites has revealed the presence of asbestos in the serpentinite and in alluvial materials downslope of the serpentinite formation.

Asbestos was found in samples from all sites except Alternative 3C-2. The presence of asbestos adds uncertainty and increases the risks involved with development of ponds at the other three sites, as the proper methods of handling and disposing the materials varies with concentration and type of asbestos. The costs associated with asbestos handling and disposal add appreciably to the costs of Alternatives 2 and 3A.

Construction at Alternative Site 2, reconstruction of Graham's Pond, which would require DSOD permitting for removal of the existing dam and construction of the new dam, does not meet the Project preference of constructing a dam that would be outside DSOD jurisdiction. This pond would also require mitigation for the loss of pond habitat. The creek that currently flows through the existing pond would have to be routed through a culvert past the new pond. Additional issues exist with constructability and potential loss of the land-owner's water rights. For these reasons, as well as the presence of asbestos-bearing serpentinite, Alternative 2 presents significant design and construction obstacles.

Construction of Alternative 3A, which would be sited in alluvial materials, would require elaborate and costly construction methods to mitigate the unstable foundation materials. The Design Report has identified removal of the fatty clay materials as a feasible approach to the geotechnical issues, and a reasonable basis for estimation of construction costs. The required deep excavation and off haul of the clay material adds significantly to the cost, and risk, of constructing a pond at Site 3A. Also, the area available for irrigation after Pond 3A construction would be 23.3 acres, slightly below the design objective of 24 acres.

Site 3C-2 is located on high ground and would require deep excavation and off-hauling of large quantities of earth. Approximately 80% of the 83,650 CY of material to be off-hauled would consist of sandstone and conglomerate, with the remainder being graywacke, shale and metashale. These materials would be suitable for use as engineered fill.

Because of the issues summarized in Table 9 and discussed above, Pond 3C-2 is the recommended Project. The next section presents the conceptual design of the recommended Project.

## Table 9: Summary of Project Alternative Pond Sites

| Site designation | 2 | 3A | 3C-2 |
|---|---|---|---|
| **Location** | Reconfiguration of Graham's Pond | Northerly of Graham's Pond | Southeasterly part of project area, top of hill |
| **Achievable Capacity** | 12.5 MG[1] | 12.5 MG | 12.5 MG |
| **Underlying Geology** | Primarily serpentinite | Alluvial materials with serpentinite beneath, poor foundation materials | Tuff and sandstone, conglomerate, graywacke and shale, |
| **Asbestos** | Found in easterly wall of existing pond | Found in alluvial sediments | None found in sandstone |
| **Groundwater** | High | High | Found at 20' below ground surface |
| **Earthwork** | 5,000 CY export of serpentinite – may contain asbestos | 51,000 CY export of clay – may contain asbestos | 83,650 CY export of sandstone, conglomerate and shale |
| **Visual Impact** | New dam would look similar to existing dam | Very visible from Graton Road | May be visible from Occidental Road & Facendini Lane |
| **Nearest Residence(s)** | 500' westerly | 300' westerly | 200' easterly; 600' northerly |
| **Affect On Land-Owners' Water Rights** | Risk of loss | None | None |
| **Nearby Wells** | None known | None known | None recorded; thought to be ~200' easterly |
| **Habitat Loss** | Pond | Stream & wetlands | Tree removal |
| **Estimate of Probable Capital Cost, million dollars[3,4]** | $ 7.2 – 7.6 | $ 10.4 – 14.4 | $ 5.0 – 5.2 |
| **Meets Project Objectives** | No[1] | No[2] | Yes |
| **Further Geotechnical Investigation Needed if Alternative is Selected** | Borings | Testing of fatty clay for asbestos | Potential groundwater investigation |
| **Risks** | Asbestos disposal | Asbestos disposal | Variable demand for engineered fill |

[1] The design capacity of 12.5 MG can be achieved at the Graham's Pond Site only by construction of a DSOD dam.
[2] The area available for irrigation after Pond 3A construction would be slightly below the design objective of 24 acres.
[3] Cost estimates do not include Graham's Pond closure, site acquisition, easements. These costs are believed to be similar for and independent of the alternative selected.
[4] The range of costs reflects the likely range of low and high costs for the disposition of excess earth materials.

# 8. RECOMMENDED PROJECT

## 8.1 GENERAL

The recommended project consists of construction of a 12.5 MG effluent storage pond at the southeast portion of the project area. The pond will have a membrane liner and a cover with collection sump and pump for rainwater. Inside slopes will be 2:1, and exterior slopes 2.5:1. A twelve foot wide access and top-of-dike road will be constructed for all-weather use. The pond will be fenced to exclude wildlife and unauthorized people. Associated infrastructure will include a transfer pump station at the WTF, extension of the existing transfer main, and an irrigation system. A new electric service will be installed at the pond to provide power for the cover sump pump and irrigation booster pump. Appendix F presents 30% conceptual design drawings for the recommended project. Drawings included in the 30% conceptual design are listed in Table 10.

**Table 10: Drawing List (30% Design) – Recommended Project**

| Sheet Number | Sheet Title |
|---|---|
| 1 | Location and Vicinity Maps and Index to Drawings |
| 2 | Abbreviations, Legend, Notes and Typical Details |
| 3 | Design Criteria, Schematic Plan and Profile |
| 4 | Overall Site Plan |
| 5 | Transfer Pump Station – Plan, Sections, Details |
| 6 | Transfer Main & Irrigation Plan (1"=40') |
| 7 | Transfer Main & Irrigation Plan (1"=40') |
| 8 | Irrigation Plan (1"=40') |
| 9 | Transfer Main & Irrigation Plan (1"=40') |
| 10 | Transfer Main & Irrigation Plan (1"=40') |
| 11 | Irrigation Booster Pump Station Plan, Sections & Details |
| 12 | Effluent Storage Pond Grading Plan |
| 13 | Effluent Storage Pond Piping Plan |
| 14 | Effluent Storage Pond Cover and Electrical Plan |
| 15 | Effluent Storage Pond Details |
| 16 | Effluent Storage Pond Cover Details |

## 8.2 SCHEMATIC PLAN AND PROFILE

Plan Sheet 4 presents design criteria and a schematic plan and profile of the proposed storage and irrigation system. The schematic shows the effluent transfer pump station at the existing WTF, the transfer main, irrigation zones, irrigation booster pump station and the effluent storage pond. The pond inlet/outlet piping, and subdrain system are shown schematically.

## 8.3 DESCRIPTION OF PROJECT ELEMENTS

### 8.3.1 Transfer Pump Station

A new 7.5 HP transfer pump station will be constructed at the WTF. Preliminary design of the transfer pump calls for installation of an above-grade polyethylene pump control tank with approximately 1,000 gallon capacity and two in-line centrifugal pumps. One pump will be duty, and one will provide mechanical redundancy. Water will flow into the pump control tank from the existing chlorine contact basins. The transfer pump will be controlled based on water level in the control tank. The transfer pump will be designed to pump at a slightly higher rate than the existing 140 gpm disinfection pump. A small concrete slab will be constructed to support the pump control tank and the pumps. Electrical and controls will be housed inside the existing treatment plant building. Disconnect switches will be provided at the pumps. A flow meter will record all flows pumped to irrigation and storage. An emergency overflow on the pump control tank will divert overflow back to the settling pond. Calculations for determining the transfer pump horsepower requirement may be found in Appendix G.

### 8.3.2 Transfer Main

The existing transfer main is a 6-inch diameter C900 PVC pipe, and ductile iron pipe for the last 200 feet before the pipe enters Graham's Pond. The 30% conceptual drawings show the new transfer main connecting to the existing C900 main near north of Occidental Road, and running approximately 3,800 feet northwesterly parallel to Occidental Road, northeasterly through the existing dam and access road to a cattle path, and then southeasterly up to the pond. Portions of the new transfer main will also serve as irrigation main (Plans, Sheet 4).

During design, a portion of the existing transfer main on the Loades property adjoining Occidental Road was exposed through pot-holing. The pot-holing allowed determination of the location of the existing main and verification that the main was constructed of C900 piping. A portion of C900 effluent main is also visible at the WTF. Since the pressure class of C900 can accommodate the planned transfer main pressure, it has been determined that the existing main between the WTF and the Loades property can be used as pressure main in the Project. Additional pot-holing will be required for two fittings on the transfer main within the Druid's cemetery, to verify thrust restraint.

### 8.3.3 Effluent Storage Pond

Plan Sheets 12, 13, and 14 show grading, piping and cover plans for the proposed pond. In the design layout for Site 3C-2, the outside pond dike height was held to a maximum of 10 feet, in order to reduce the site footprint and minimize tree loss. The maximum operating water depth will be approximately 37 feet, and side slopes will be 2.5:1 for exterior dikes and 2:1 inside the pond. A 10 foot setback was held from the adjoining property line. The elevation for the top of dike was set so that the dike will not obstruct the existing line of sight from the adjoining parcel on the east.

Construction will involve excavation to depths up to 55 feet, in sandstone, conglomerate, keratophyre and quartz keratophyre tuff (tuff), shale, graywacke and metashale. The pond has been sited to lie outside the zone of ultramafic rock. The geotechnical investigation indicates that the pond will be excavatable using standard construction equipment. Stripped topsoil may be spread on the land-owners property. Excavated bedrock conglomerate materials will be suitable for use in

construction of fill. The remainder of the excavated material, approximately 83,650 CY of excess earth, will be off-hauled from the site.

The pond will be lined with an HDPE membrane to prevent seepage of effluent to groundwater.. A non-woven geotextile fabric placed between the excavated pond and the liner will protect the liner from abrasion and puncture by the soil and rock substrate. Sheet 15 of the Plans includes details of the liner construction.

The pond will be covered with a synthetic membrane to exclude rainwater. The cover will be made of Hypalon, which is highly resistant to UV degradation. The cover will float on the water in the pond, and will be designed to lie upon the sides and bottom of the pond when it is empty. As the pond fills and the cover floats, a system of floats and weights in the cover will create channels that lead to a sump. A centrifugal pump will remove rainwater from the pond. The water will be directed to a swale(s) to be constructed in the area southerly of the pond, between the access road and the property line. The swales will be designed in compliance with requirements implemented by the Regional Board under the Statewide Construction General Stormwater Discharge Permit. Sheet 16 of the Plans includes details of the pond cover construction.

A system of subdrains will collect groundwater that may accumulate seasonally around the pond and convey it to an outlet to surface drainage.

Effluent will flow into and out of the pond through a single bottom inlet/outlet. Plan Sheet 15 shows the detail of the inlet-outlet.

A home is located approximately 200 feet to the east of the site. Although there is no record of a well, it is believed that a drinking water well was constructed approximately 130 feet from the parcel line. The next phase of the project will include determining the location of any active drinking water well(s) on the adjacent parcel and verifying whether the pond would meet the Title 22 required 100-foot setback. Because the pond will be lined, the risk of seepage of stored effluent to any receptor will be exceptionally low. CEQA analysis will include an analysis of potential changes to the water table and corollary impacts to adjacent wells. Design mitigations will be implemented if necessary.

The forested area impacted by Alternative 3C-2 includes Douglas fir and redwood trees, considered timber species by the California Department of Forestry (CDF). Any conversion of timber to non-timber uses requires a permit from CDF. Since the impacted area is less than three acres, preparation of a Less Than 3 Acre Conversion Exemption will be required. A Sonoma County zoning permit will also be required for the conversion. However, the County does not require mitigation for tree removal under this Exemption.

### 8.3.4 Access Road

The existing farm road to Facendini Lane would be reconstructed to provide a 12-foot wide all-season access road to the pond site, with a turn-off onto the farm road to the northwest. County parcel maps indicate that the access road, in the alignment shown on the 30% conceptual design figures and plans, leads directly onto County property without crossing any privately-owned parcel. Title to the land outside the access gate would be verified in the next phase of design.

### 8.3.5 Disturbance of Ultramafic Rock Areas

Although the pond itself lies outside the area of serpentinite rock, the project will disturb serpentine during road construction and trenching operations. Dust control requirements of the CARB will be

adhered to. A 3-inch cover of non-ultramafic earth will be placed over all areas disturbed during construction.

### 8.3.6 Irrigation System

Proposed irrigation areas and distribution mains are shown on Plan Sheet 4, Appendix F. Irrigation areas will be in two distinct pressure zones. The lower zone, which includes approximately 18 acres, will be irrigated by gravity from the pond. A small pump station will provide pressure for irrigating approximately six acres in the upper zones.

The irrigation system has been designed to provide a permanent backbone of distribution mains, with connections for portable laterals. There will be seven subareas in the lower pressure zone and five in the upper pressure zone. The irrigation system has been sized to be able to accommodate all the water stored in the reservoir during the design peak rainfall year. Following peak rainfall seasons, each subarea is designed to be irrigated five to eight hours a day, one day a week, during the peak irrigation months, and proportionally less during the spring and late season, in order to empty the reservoir. In irrigation seasons following more typical rainfall seasons, irrigation could be managed to irrigate fewer subareas, to irrigate fewer days per week, and/or to irrigate fewer hours per day.

Irrigation water will be transmitted to the irrigation areas through buried PVC pipe, 6-inch and 4-inch in diameter. Laterals will be portable 3-inch aluminum latch pipe. Connections for the lateral lines will be spaced at 60 ft intervals along the distribution mains. Three-quarter-inch impact sprinkler heads will be spaced at 40 ft intervals along the lateral lines. The same type of sprinkler head may be used for the entire system. Use of two or three different types of nozzles will allow for the heads to be used at pressures ranging 40-80 psi and for flow rates ranging 8-10 gpm per sprinkler head.

In the low pressure zone, the seven irrigation subareas range in size from approximately 2.5-2.8 acres. Each subarea will be irrigated with 100,000 – 120,000 gallons one time per week, at a flow rate of approximately 275 gpm. A pressure reducing valve will ensure that the pressure will not exceed 80 psi at any of the sprinkler heads in the low zone.

The upper pressure zone, five irrigation subareas range from 1.1 – 1.5 acres and each will be irrigated with 50,000 – 65,000 gallons of effluent one time per week. A 7.5 HP pump will supply 145 gpm to the zone. Water will be transported from the pump station easterly up to a maximum elevation of approximately 890 ft, or westerly down to a minimum elevation of approximately 760 ft. A pressure reducing valve will be placed in the line which transports the water westerly to ensure the pressure at the lowest point of the upper pressure zone does not exceed 80 psi.

It will also be possible to irrigate the lower pressure zone directly from the treatment plant. It will be possible to irrigate one or both of the two pressure zones at any given time. Calculations used in developing the conceptual design of the irrigation system may be found in Appendix G.

## 8.4 ESTIMATE OF COSTS

The cost estimate for the recommended project was presented in Section 7, Table 8. The estimated total capital cost is in the range of $5.0 to $5.2 million. The range of costs reflects the likely range of low and high costs for the disposition of excess earth materials. The cost estimate does not include potential restoration of Graham's Pond or costs for easement, lease or purchase of the site for pond construction and development of the irrigation system.

## 8.5 RECOMMENDATIONS

It is recommended that the District pursue construction of the recommended pond project, with associated transfer and irrigation facilities. The next phase of work will include the following areas which have been identified as needing further exploration:

- Determination of the location of any active drinking water well(s) on the adjacent parcel and verification that the pond would meet the Title 22 required 100-foot setback. Because the pond will be lined, the risk of seepage of stored effluent to any receptor will be exceptionally low.
- Development of an agreement with the landowners for construction of the recommended project. The agreement might take the form of an easement, lease or purchase. Financial arrangements would be negotiated, based upon an appraisal of the land value. The agreement with the landowners should be completed only after completion of CEQA and NEPA reviews.
- Title verification for parcel outside access gate to Facendini Lane.
- Additional pot-holing will be required for two fittings on the transfer main within the Druid's cemetery, to verify thrust restraint.

# APPENDIX A

## TECHNICAL MEMORANDUM NO. 1

## WATER BALANCE PARAMETERS AND OUTPUTS

Brelje & Race
CONSULTING CIVIL ENGINEERS

# TECHNICAL MEMORANDUM NO. 1

TO: Douglas Messenger

FROM: Richard Ingram RI

SUBJECT: **OCSD Wastewater Storage and Reclamation Project**
**Water Balance Parameters & Outputs**
B&R File No. 3619.02

DATE: January 5, 2012

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## INTRODUCTION

As part of the Occidental County Sanitation District (District) Wastewater Storage and Reclamation 30% Conceptual Design, a water balance model was developed of the District's collection, treatment, storage and reclamation system. The water balance model calculates the irrigation area and storage capacity required for reliable operation at potential future conditions.

The model tracks inflows and outflows to the reclamation system on a month-by-month basis over a twelve month period. Water enters the system as wastewater, as inflow and infiltration into the collection system, and as rainfall directly on uncovered ponds. Water may leave the zero-discharge Occidental system as irrigation and as evaporation and percolation from ponds. For the system to balance, inflows must equal outflows for the year. Water balance calculations are iterated, with adjustments to storage and irrigation capacities, until the capacities are adequate for year-end storage volume to equal the year-beginning storage volume. Storage capacity must be sufficient for the accumulation of wet season influent flows, and irrigation area must be sufficient to take up the entire year's accumulation of influent flow during the dry season.

Selection of input parameters strongly influences the water balance model's results. Input parameters have been selected to reflect real conditions as much as reasonably possible. Parameters have been selected to be conservative enough so that the system would not be undersized. However, an effort has been made to avoid inputs so conservative that the system would be over-sized, as this would result in excessive capital costs and inefficient use of resources. A combination of data review, statistical analysis and engineering judgment are used in selection of input parameters.

The actual capacity of the reservoir that can be built may be limited by constraints of geology, geomorphology and budget. The results of the water balance calculations can then be used to assess the Average Dry Weather Flow and rainfall year recurrence capacity associated with the constructible reservoir.

Parameters utilized in the water balance model are described in the matrix on the following pages, followed by a summary of model outputs and graphs of historical data and input parameters.

## INPUT PARAMETERS FOR WATER BALANCE

| Parameter | Value | Discussion |
|---|---|---|
| Average Dry Weather Flow (ADWF) | 26,000 gallons per day (gpd) | See Figure 1 for a graph of historical ADWF. The District identified the existing ADWF as 19,000 gpd in the Feasibility Study for CDO Compliance Project. 19,000 gpd is both the maximum and the 95th centile value for 1995-2011.<br>The District then added 4,200 gpd for the planned Harmony Village development, plus a 10% safety factor, for a design ADWF of 26,000 gpd.<br>For comparison, the 2002 Engineering Report for Camp Meeker and the District projected a future ADWF of 25,000 gpd for the District, allowing for development of currently vacant property, and assuming Harmony School to remain.<br>The projected ADWF value of 26,000 gpd was deemed appropriate for the current project. |
| Weather year | October-September | Storage pond(s) need to be functionally empty at the end of September to be ready to accommodate the wet season's influent flows. |
| Rainfall | Total annual rainfall for varying recurrence intervals projected using Pearson distribution. | Figure 2 presents a histogram of historical annual rainfall in Occidental. The historical maximum precipitation year is 1982-83, with 102.6 inches measured. Pearson distribution allows projection of 90-, 100- and 200-year values from the existing 70 years of data. Annual rainfall values based on the Pearson distribution and used in the model may be found in Table 1. |
| | Design for 100-year recurrence (99th centile) total annual rainfall | The 100-year recurrence interval is recognized as an industry standard, reflecting adequate caution. The probability of a failure is small, and the impact of a failure would be a potential discharge that did not meet Regional Board requirements, not a threat to human life. |
| | Distribution of annual rainfall based on historical average monthly pattern | Use of average rainfall distribution is customary in water balance models. |
| | Direct rainfall falls onto one acre at treatment plant. | The District's estimate of one acre pond drainage area was verified by B&R. |

| Parameter | Value | Discussion |
|---|---|---|
| Rainfall, cont'd. | No direct rainfall onto storage pond | It has been previously determined that it would be impracticable to construct a pond with no cover. A pond without a cover would require an increased capacity to accommodate the direct input of rainfall into the pond. The larger pond would be under DSOD jurisdiction unless the dam height was less than 15 feet. Siting of a shallower pond would be challenging in the hilly terrain of the project area. Further, the shallower pond would have a greater surface area and therefore collect more direct rainfall, and therefore need to be larger still. It was estimated that an uncovered pond would have to have approximately two times more capacity than a covered pond.<br>It was therefore assumed storage pond will be covered. |
| Inflow and Infiltration (I&I) | Determine I&I from relationship of historical total annual influent flow minus ADWF as function of rainfall, for years since recent sewer improvements were completed (weather years 2008-09 through 2010-11). | Figure 3 presents monthly flows and rainfall for 2003 through 2011. Years 2008-09 through 2010-11 are the only full weather years since completion of the improvements of the sewer system in the autumn of 2008. Figures 4 and 5 show a plot of annual I&I as a function of rainfall, with natural logarithm and straight line best fit lines.<br>The natural logarithm relationship reflects that there is virtually no I&I when there is less than 20 inches total annual rainfall, and that I&I starts to level off at more than 80 inches total annual rainfall.<br>The straight line relationship indicates I&I would go to approximately zero when there was zero rainfall.<br>The fit for logarithmic and straight fit lines are both fairly good. The straight line relationship was selected for projecting I&I in the water balance model because it gives a more conservative projection for peak rainfall year I&I.<br>Note the model ties I&I exclusively to rainfall. The implicit assumptions are that (1) the sewers will not be extended significantly and (2) sewer maintenance will be performed to prevent increases in I&I. |
| | Monthly distribution of I&I based on District's historical patterns. | See Figure 6. Historical distribution was adjusted to shift I&I from July & August into January. This adjustment puts a larger portion of the annual I&I into the winter months, thereby increasing the storage requirement. |

| Parameter | Value | Discussion |
|---|---|---|
| Evaporation & percolation losses | Losses from existing aerated and settling ponds included in model.<br>Surface area measured as approximately 0.45 acres.<br>Assume evaporation at historical average pan evaporation rate at Lake Sonoma station.<br>Assume percolation at $1X10^{-6}$ cm/s. | The monthly volume of sewage pumped into the treatment plant was compared to monthly transfer volume, for summer months 2009-11. The loss to evaporation was estimated based on measured pan evaporation at Lake Sonoma for the particular month. The remaining loss was then calculated, and was found to be in the range of $4X10^{-6}$ to $1X10^{-5}$ cm/s. This loss is attributed to percolation plus additional evaporation due to splash aerators. The rate of evaporation plus percolation used in the model therefore appears less than actual, and is deemed conservative. |
| | No losses from storage pond | The pond will be covered and lined. |
| Surface Water Discharge | Zero | The District has determined that it will aim for elimination of the current discharge to Dutch Bill Creek. |
| Irrigation Usage | Total annual use of 27 inches | The irrigation of 27" per year is believed to be appropriate for pasture or fodder crops in the Occidental area. In the absence of data from reclamation irrigation in the coastal hills, this estimate of irrigation usage was extrapolated from usage inland and on the coast. In the Town of Windsor reclamation system, on the Santa Rosa plain, historical records indicate annual use of 30" on pasture. In a review of usage at Bodega Bay Public Utility District, B&R estimated that the golf course uses approximately 24" per year of reclaimed water. The value of 27" was selected for Occidental as midway between the coastal and Santa Rosa plain values.<br>If the water is used for irrigating vineyards, the required area would be approximately five to ten times greater. |
| | Monthly usage pattern based on District's historical use pattern, with autumn usage reduced. | See Figure 7. Reduction of autumn irrigation increases the requirement for winter storage, making this a conservative change. |

## WATER BALANCE MODEL OUTPUTS

Water balance model outputs for the projected ADWF of 26,000 gpd, and for the current ADWF of 19,000 gpd, are summarized in Table 1. Copies of the water balance tabulations for the 100-year (99 percentile) rainfall year are attached.

**Table 1: Summary of Water Balance Outputs**

| Recurrence Interval, years | Total Annual Rainfall Percentile Ranking | Total Annual Rainfall, inches | Inflow and Infiltration, MG | ADWF of 26,000 gpd | | ADWF of 19,000 gpd | |
|---|---|---|---|---|---|---|---|
| | | | | Required Storage Capacity, MG | Required Irrigation Area, acres | Required Storage Capacity, MG | Required Irrigation Area, acres |
| 200 | 99.5% | 119.3 | 5.56 | 13.1 | 25 | 11.7 | 22 |
| 100 | 99% | 110.3 | 5.14 | 12.5 | 24 | 11.1 | 21 |
| 20 | 95% | 88.0 | 4.13 | 11.0 | 22 | 9.7 | 19 |
| 10 | 90% | 78.2 | 3.68 | 10.3 | 21 | 9.1 | 17 |
| 5 | 80% | 67.1 | 3.18 | 9.5 | 20 | 8.4 | 16 |
| 2 | 50% | 49.6 | 2.38 | 8.4 | 18 | 7.2 | 14 |
| 1.010101 | 1% | 19.6 | 1.01 | 6.4 | 15 | 5.3 | 11 |

MG - million gallons

As can be seen in Table 1, the projected storage and irrigation capacities for the future ADWF in the 100-year recurrence rainfall year are 12.5 MG and 24 acres, respectively. These future 100-year rainfall requirements are significantly greater than the current median rainfall year requirements for 6.3 MG storage and 13 acres of irrigation area.

## FIGURES

**Figure 1**

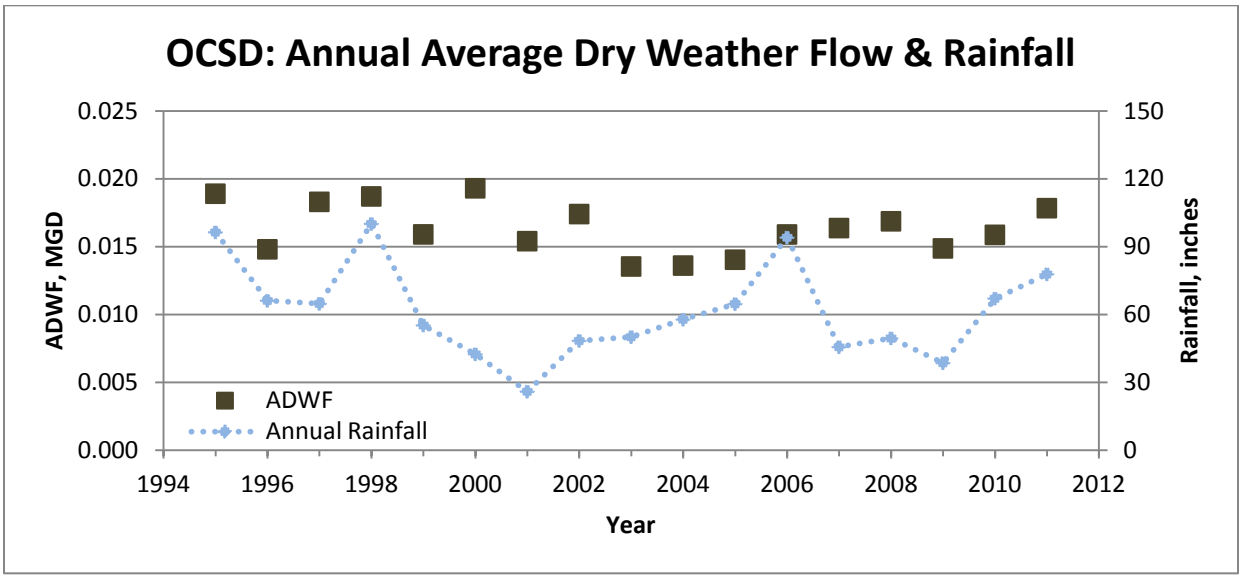


**Figure 2**

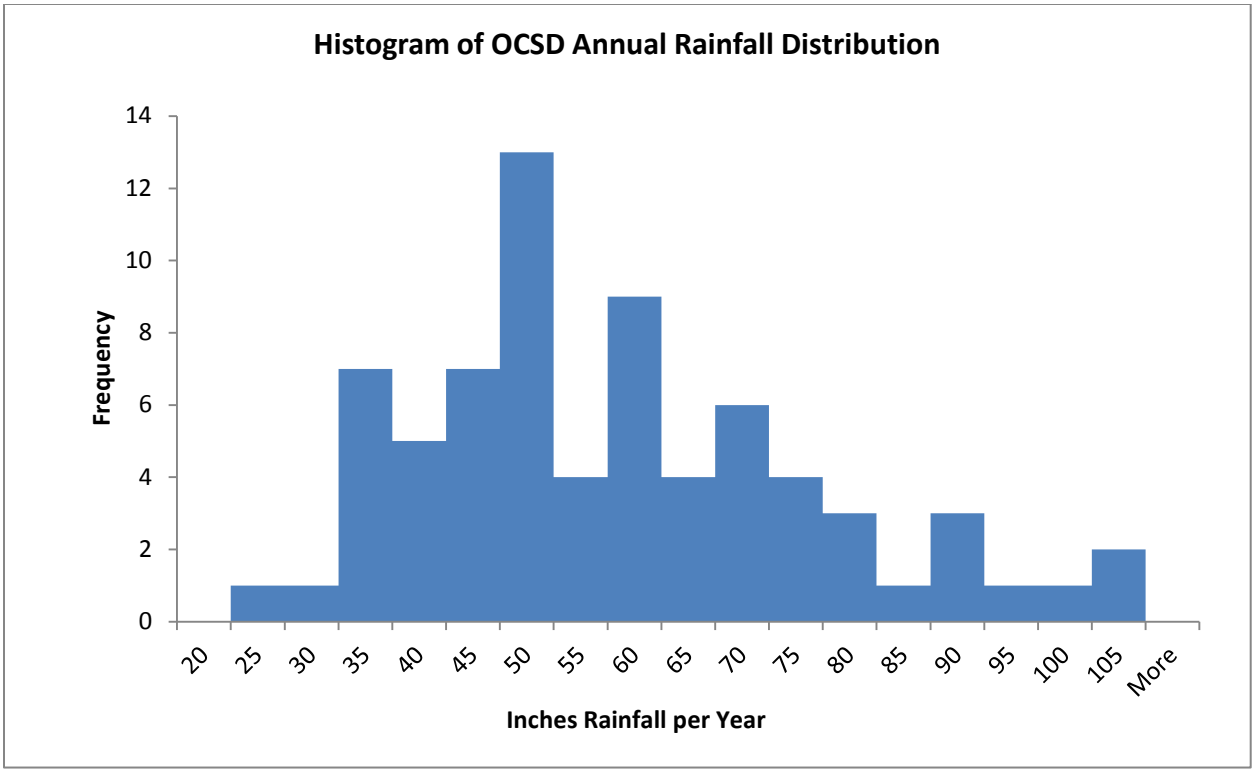



**Figure 3**

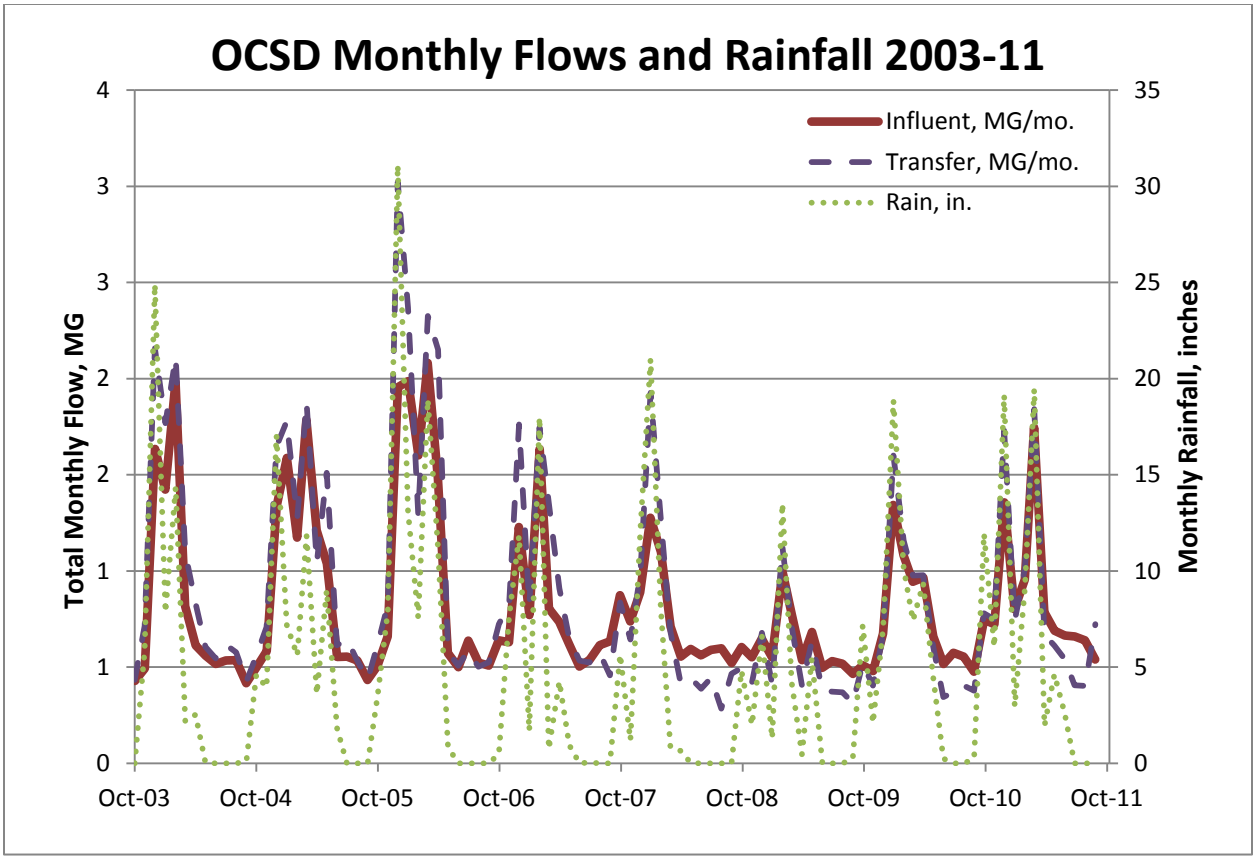


**Figure 4**

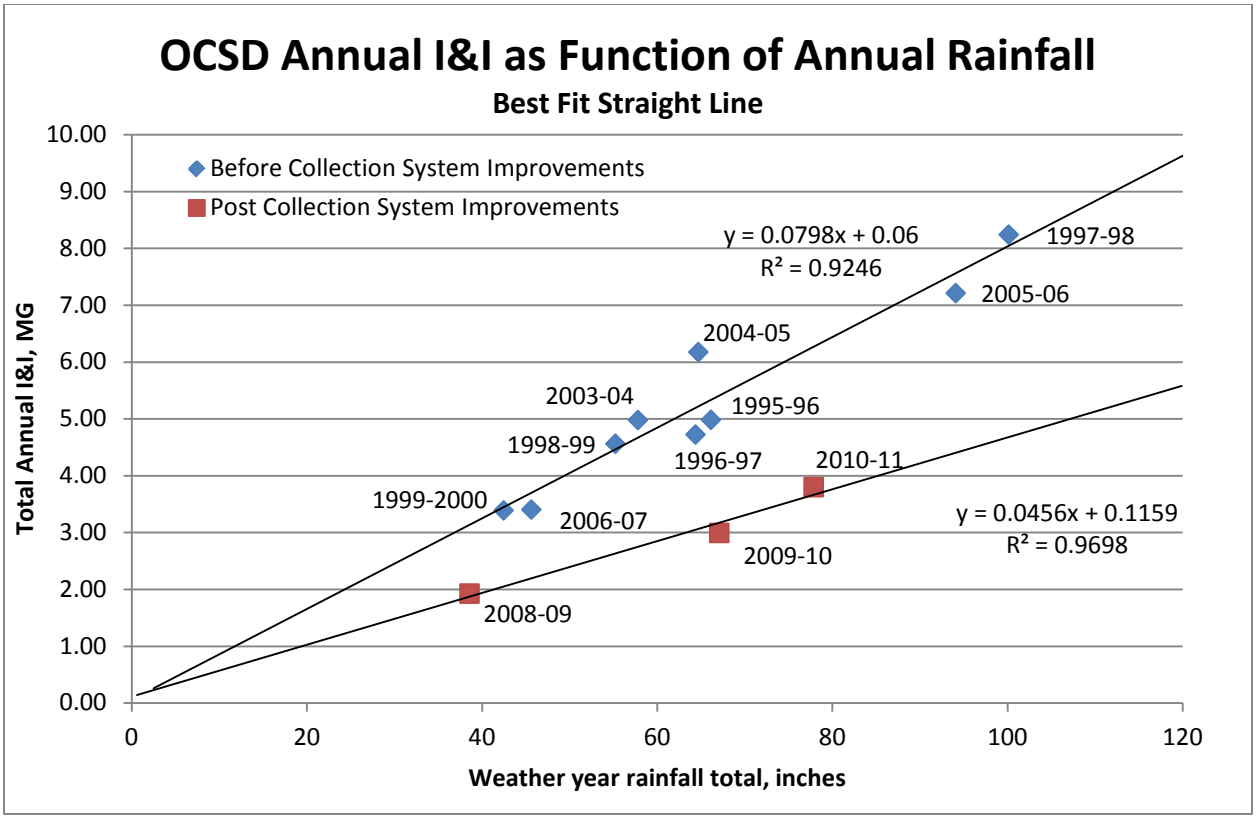



**Figure 5**

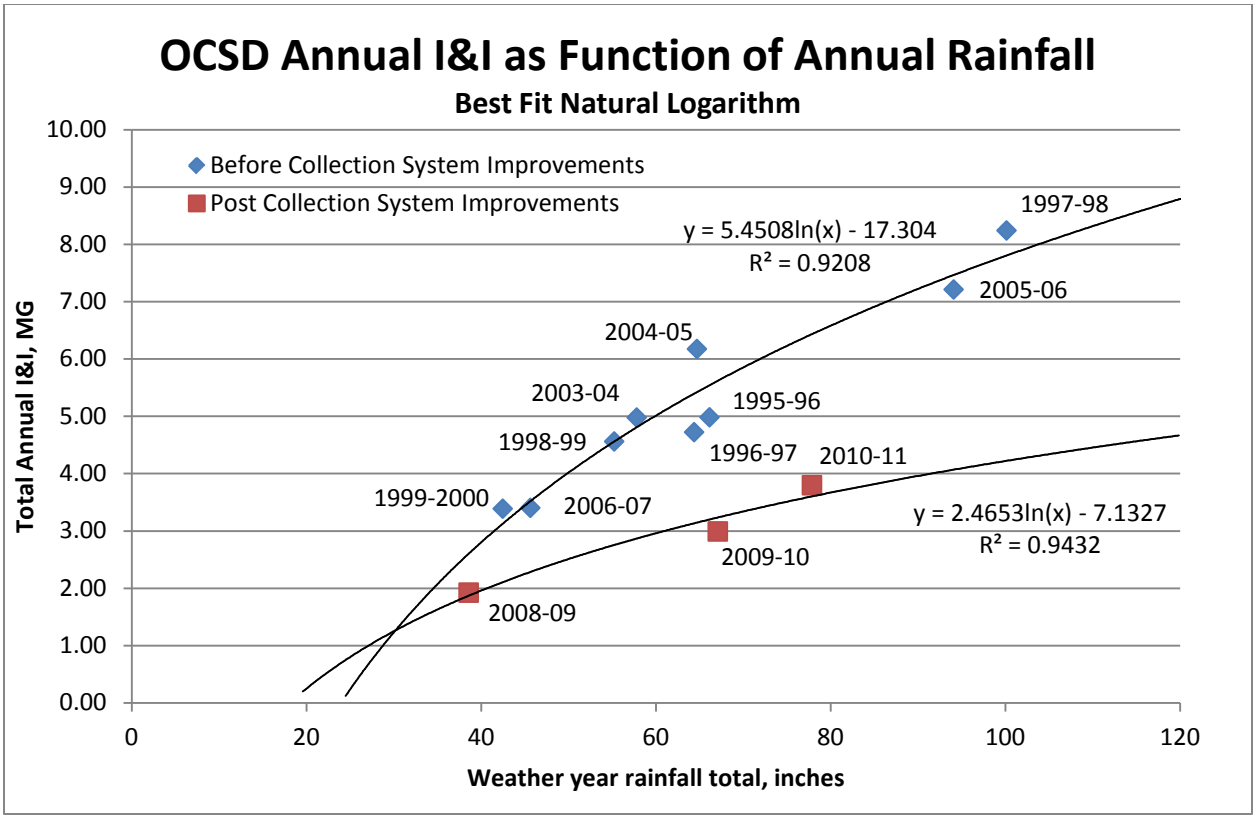


**Figure 6**

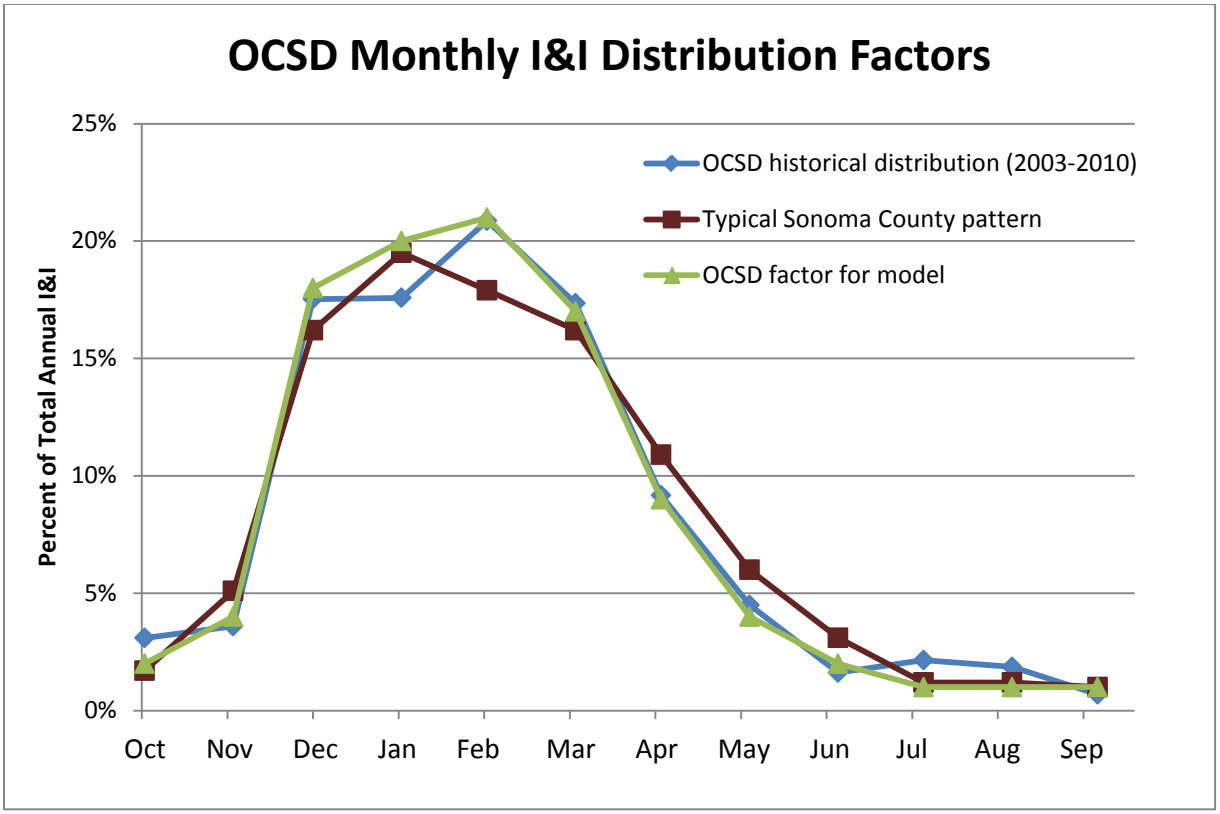
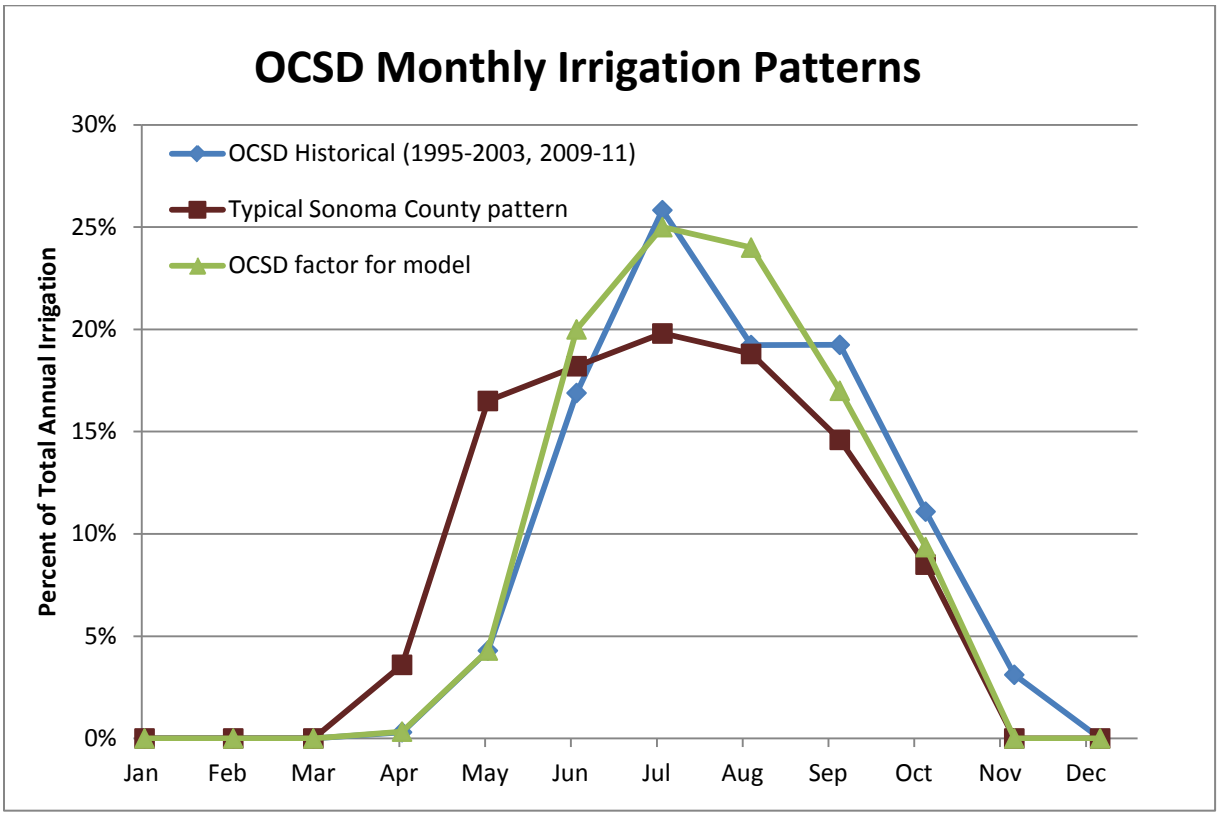



**Figure 7**



**ATTACHMENTS**

Water Balance Tabulation for 26,000 gpd ADWF and 100-year Recurrence Annual Total Rainfall

Water Balance Tabulation for 26,000 gpd ADWF and 100-year Recurrence Annual Total Rainfall

# Occidental County Sanitation District
# Water Reclamation Treatment, Storage and Disposal
# Water Balance Analysis

I&I based on Linear Best Fit of Annual Data since Sewer Repairs Completed in 2008

## User Input

| | | |
|---|---|---|
| Treatment Plant Influent Flow, ADWF (MGD) = | 0.026 | PROJECTED AVERAGE DRY WEATHER FLOW |
| Percent Rainfall Year (100%=wettest, 0%=dryest) = | 99.0% | |
| Annual Rainfall (in) = | 110.27 | |
| Average Irrigation Rate (inches per year) = | 27 | |
| Assumed Storage Capacity (MG) = | 12.5 | pond |
| Dead Storage (percent of total) = | 5% | |
| Storage Pond Permeability (centimeters per second) = | 0.00E+00 | (lined) |
| Assumed Storage Pond Evaporation Area (acres per MG) = | 0.0000 | (covered) |
| Assumed Storage Pond Direct Rainfall Area (acres per MG) = | 0.0000 | (covered) |
| Treatment Pond Permeability (centimeters per second) = | 1.E-06 | |
| Treatment Pond Evaporation Area (acres) = | 0.45 | |
| Treatment Plant Direct Rainfall Area (acres) = | 1.0 | |

## Model Output

| | |
|---|---|
| Total Irrigation Volume÷Average Irrigation Rate (acres) = | 22.8 |
| **Required Irrigation Area (acres) =** | **24** |
| **Required Storage Volume (MG) =** | **12.5** |

| | --IN-- | | | | --OUT-- | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Month | ADWF Influent | Infiltration and Inflow | Direct Rainfall to Plant | Total Input | Treatment Pond Percolation | Treatment Pond Evaporation | Net Storage Available for Irrigation | Irrigation | End of Month Storage |
| | | | | | | | | | 0.63 |
| Oct | 0.81 | 0.10 | 0.19 | 1.10 | 0.01 | 0.05 | 1.66 | 1.03 | 0.63 |
| Nov | 0.78 | 0.21 | 0.40 | 1.39 | 0.01 | 0.03 | 1.98 | 0.00 | 1.98 |
| Dec | 0.81 | 0.93 | 0.47 | 2.20 | 0.01 | 0.02 | 4.15 | 0.00 | 4.15 |
| Jan | 0.81 | 1.03 | 0.64 | 2.48 | 0.01 | 0.02 | 6.59 | 0.00 | 6.59 |
| Feb | 0.73 | 1.08 | 0.49 | 2.29 | 0.01 | 0.03 | 8.85 | 0.00 | 8.85 |
| Mar | 0.81 | 0.87 | 0.45 | 2.13 | 0.01 | 0.05 | 10.92 | 0.00 | 10.92 |
| Apr | 0.78 | 0.46 | 0.25 | 1.49 | 0.01 | 0.07 | 12.33 | 0.06 | 12.27 |
| May | 0.81 | 0.21 | 0.07 | 1.08 | 0.01 | 0.09 | 13.24 | 0.76 | 12.49 |
| Jun | 0.78 | 0.10 | 0.01 | 0.89 | 0.01 | 0.11 | 13.26 | 3.52 | 9.74 |
| Jul | 0.81 | 0.05 | 0.01 | 0.87 | 0.01 | 0.12 | 10.48 | 4.40 | 6.08 |
| Aug | 0.81 | 0.05 | 0.01 | 0.87 | 0.01 | 0.10 | 6.84 | 4.22 | 2.61 |
| Sep | 0.78 | 0.05 | 0.01 | 0.84 | 0.01 | 0.08 | 3.35 | 2.73 | 0.63 |
| Totals | 9.49 | 5.14 | 3.00 | 17.63 | 0.15 | 0.75 | | 16.72 | |

All values in million gallons (MG).

## Model Assumptions

| Month | Irrigation Distribution Factor | Evaporation (inches) | I/I (MG) | Rainfall (inches) | Rainfall Distribution Factor |
|---|---|---|---|---|---|
| Oct | 9.4% | 4.42 | 0.10 | 6.95 | 6.3% |
| Nov | 0.0% | 2.17 | 0.21 | 14.89 | 13.5% |
| Dec | 0.0% | 1.54 | 0.93 | 17.31 | 15.7% |
| Jan | 0.0% | 1.50 | 1.03 | 23.71 | 21.5% |
| Feb | 0.0% | 2.20 | 1.08 | 17.86 | 16.2% |
| Mar | 0.0% | 3.73 | 0.87 | 16.54 | 15.0% |
| Apr | 0.3% | 5.68 | 0.46 | 9.04 | 8.2% |
| May | 4.3% | 7.40 | 0.21 | 2.43 | 2.2% |
| Jun | 20.0% | 8.63 | 0.10 | 0.44 | 0.4% |
| Jul | 25.0% | 9.52 | 0.05 | 0.44 | 0.4% |
| Aug | 24.0% | 8.32 | 0.05 | 0.44 | 0.4% |
| Sep | 17.0% | 6.63 | 0.05 | 0.22 | 0.2% |
| Totals | 100.0% | 61.74 | 5.14 | 110.27 | 100.00% |

## OUTPUT SUMMARY

| FOR ADWF = 26,000 GPD (PROJECTED FUTURE) | | | | |
|---|---|---|---|---|
| Rain Centile | Rain | I&I | Pond | Irri @ 27" |
| 99.5% | 119.3 | 5.56 | 13.1 | 25 |
| 99% | 110.3 | 5.14 | 12.5 | 24 |
| 95% | 88.0 | 4.13 | 11.0 | 22 |
| 90% | 78.2 | 3.68 | 10.3 | 21 |
| 80% | 67.1 | 3.18 | 9.5 | 20 |
| 50% | 49.6 | 2.38 | 8.4 | 18 |
| 1% | 19.6 | 1.01 | 6.4 | 15 |

| FOR ADWF = 19,000 GPD (CURRENT MAX) | | | | |
|---|---|---|---|---|
| Rain Centile | Rain | I&I | Pond | Irri @ 27" |
| 99.50% | 119.3 | 5.56 | 11.7 | 22 |
| 99% | 110.3 | 5.14 | 11.1 | 21 |
| 95% | 88.0 | 4.13 | 9.7 | 19 |
| 90% | 78.2 | 3.68 | 9.1 | 17 |
| 80% | 67.1 | 3.18 | 8.4 | 16 |
| 50% | 49.6 | 2.38 | 7.2 | 14 |
| 1% | 19.6 | 1.01 | 5.3 | 11 |

NOTES

1 Irrigation distribution factor based upon observed operations in OCSD.

2 Evaporation is long term average pan evaporation at Lake Sonoma.

3 Rainfall from 70 years of NOAA data for Occidental. Distribution of rainfall for weather years with less than 78 inches total rain is based on the average distribution. Distribution of rainfall for weather years with more than 78 inches total rain is based on the distribution for years with more than 78 inches rainfall.

# Occidental County Sanitation District
# Water Reclamation Treatment, Storage and Disposal
# Water Balance Analysis

I&I based on Linear Best Fit of Annual Data since Sewer Repairs Completed in 2008

## User Input

| | | |
|---|---|---|
| Treatment Plant Influent Flow, ADWF (MGD) = | 0.019 | CURRENT AVERAGE DRY WEATHER FLOW |
| Percent Rainfall Year (100%=wettest, 0%=dryest) = | 99.0% | |
| Annual Rainfall (in) = | 110.27 | |
| Average Irrigation Rate (inches per year) = | 27 | |
| Assumed Storage Capacity (MG) = | 12.5 | pond |
| Dead Storage (percent of total) = | 5% | |
| Storage Pond Permeability (centimeters per second) = | 0.00E+00 | (lined) |
| Assumed Storage Pond Evaporation Area (acres per MG) = | 0.0000 | (covered) |
| Assumed Storage Pond Direct Rainfall Area (acres per MG) = | 0.0000 | (covered) |
| Treatment Pond Permeability (centimeters per second) = | 1.E-06 | |
| Treatment Pond Evaporation Area (acres) = | 0.45 | |
| Treatment Plant Direct Rainfall Area (acres) = | 1.0 | |

## Model Output

| | |
|---|---|
| Total Irrigation Volume÷Average Irrigation Rate (acres) = | 19.3 |
| **Required Irrigation Area (acres) =** | **21** |
| **Required Storage Volume (MG) =** | **11.1** |

| | --IN-- | | | | --OUT-- | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Month | ADWF Influent | Infiltration and Inflow | Direct Rainfall to Plant | Total Input | Treatment Pond Percolation | Treatment Pond Evaporation | Net Storage Available for Irrigation | Irrigation | End of Month Storage |
| | | | | | | | | | 0.63 |
| Oct | 0.59 | 0.10 | 0.19 | 0.88 | 0.01 | 0.05 | 1.44 | 0.81 | 0.63 |
| Nov | 0.57 | 0.21 | 0.40 | 1.18 | 0.01 | 0.03 | 1.77 | 0.00 | 1.77 |
| Dec | 0.59 | 0.93 | 0.47 | 1.99 | 0.01 | 0.02 | 3.72 | 0.00 | 3.72 |
| Jan | 0.59 | 1.03 | 0.64 | 2.26 | 0.01 | 0.02 | 5.95 | 0.00 | 5.95 |
| Feb | 0.53 | 1.08 | 0.49 | 2.10 | 0.01 | 0.03 | 8.01 | 0.00 | 8.01 |
| Mar | 0.59 | 0.87 | 0.45 | 1.91 | 0.01 | 0.05 | 9.86 | 0.00 | 9.86 |
| Apr | 0.57 | 0.46 | 0.25 | 1.28 | 0.01 | 0.07 | 11.06 | 0.05 | 11.01 |
| May | 0.59 | 0.21 | 0.07 | 0.86 | 0.01 | 0.09 | 11.77 | 0.66 | 11.10 |
| Jun | 0.57 | 0.10 | 0.01 | 0.68 | 0.01 | 0.11 | 11.67 | 3.08 | 8.59 |
| Jul | 0.59 | 0.05 | 0.01 | 0.65 | 0.01 | 0.12 | 9.11 | 3.85 | 5.26 |
| Aug | 0.59 | 0.05 | 0.01 | 0.65 | 0.01 | 0.10 | 5.80 | 3.70 | 2.10 |
| Sep | 0.57 | 0.05 | 0.01 | 0.63 | 0.01 | 0.08 | 2.64 | 2.01 | 0.63 |
| Totals | 6.94 | 5.14 | 3.00 | 15.07 | 0.15 | 0.75 | | 14.17 | |

All values in million gallons (MG).

## Model Assumptions

| Month | Irrigation Distribution Factor | Evaporation (inches) | I/I (MG) | Rainfall (inches) | Rainfall Distribution Factor |
|---|---|---|---|---|---|
| Oct | 9.4% | 4.42 | 0.10 | 6.95 | 6.3% |
| Nov | 0.0% | 2.17 | 0.21 | 14.89 | 13.5% |
| Dec | 0.0% | 1.54 | 0.93 | 17.31 | 15.7% |
| Jan | 0.0% | 1.50 | 1.03 | 23.71 | 21.5% |
| Feb | 0.0% | 2.20 | 1.08 | 17.86 | 16.2% |
| Mar | 0.0% | 3.73 | 0.87 | 16.54 | 15.0% |
| Apr | 0.3% | 5.68 | 0.46 | 9.04 | 8.2% |
| May | 4.3% | 7.40 | 0.21 | 2.43 | 2.2% |
| Jun | 20.0% | 8.63 | 0.10 | 0.44 | 0.4% |
| Jul | 25.0% | 9.52 | 0.05 | 0.44 | 0.4% |
| Aug | 24.0% | 8.32 | 0.05 | 0.44 | 0.4% |
| Sep | 17.0% | 6.63 | 0.05 | 0.22 | 0.2% |
| Totals | 100.0% | 61.74 | 5.14 | 110.27 | 100.00% |

## OUTPUT SUMMARY

| FOR ADWF = 26,000 GPD (PROJECTED FUTURE) | | | | |
|---|---|---|---|---|
| Rain Centile | Rain | I&I | Pond | Irri @ 27" |
| 99.5% | 119.3 | 5.56 | 13.1 | 25 |
| 99% | 110.3 | 5.14 | 12.5 | 24 |
| 95% | 88.0 | 4.13 | 11.0 | 22 |
| 90% | 78.2 | 3.68 | 10.3 | 21 |
| 80% | 67.1 | 3.18 | 9.5 | 20 |
| 50% | 49.6 | 2.38 | 8.4 | 18 |
| 1% | 19.6 | 1.01 | 6.4 | 15 |

| FOR ADWF = 19,000 GPD (CURRENT MAX) | | | | |
|---|---|---|---|---|
| Rain Centile | Rain | I&I | Pond | Irri @ 27" |
| 99.50% | 119.3 | 5.56 | 11.7 | 22 |
| 99% | 110.3 | 5.14 | 11.1 | 21 |
| 95% | 88.0 | 4.13 | 9.7 | 19 |
| 90% | 78.2 | 3.68 | 9.1 | 17 |
| 80% | 67.1 | 3.18 | 8.4 | 16 |
| 50% | 49.6 | 2.38 | 7.2 | 14 |
| 1% | 19.6 | 1.01 | 5.3 | 11 |

NOTES

1 Irrigation distribution factor based upon observed operations in OCSD.

2 Evaporation is long term average pan evaporation at Lake Sonoma.

3 Rainfall from 70 years of NOAA data for Occidental. Distribution of rainfall for weather years with less than 78 inches total rain is based on the average distribution. Distribution of rainfall for weather years with more than 78 inches total rain is based on the distribution for years with more than 78 inches rainfall.

# APPENDIX B

## TECHNICAL MEMORANDUM NO. 2

## SUMMARY OF REGULATORY LIMITATIONS

## MATRIX OF AGENCY REQUIREMENTS

Brelje & Race
CONSULTING CIVIL ENGINEERS

# TECHNICAL MEMORANDUM NO. 2 – REVISED

TO: Douglas Messenger

FROM: Richard Ingram RI

SUBJECT: **OCSD Wastewater Storage and Reclamation Project**
**Applicable Regulatory Limitations and Permitting Requirements**
B&R File No. 3619.02

DATE: March 13, 2012

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## INTRODUCTION

As part of the Occidental County Sanitation District (District) Wastewater Storage and Reclamation 30% Conceptual Design and Environmental Information Document, a review has been made of potentially applicable regulations and associated permitting requirements. The intent of the review was to determine regulatory limitations on the siting and construction of the proposed reservoir and irrigation areas, to form a project response to the limitations, to identify permitting requirements, and direct the design criteria. Areas of review included portions of the California Water Code, Title 22, Title 27, the North Coast Regional Water Quality Control Board (Regional Board) Water Quality Control Plan (*Basin Plan*), Waste Discharge Requirements recently issued by the Regional Board, Sections 404 and 401 of the Federal Clean Water Act, Section 7 of the Endangered Species Act, and the State of California Fish and Game code.

Regulations are summarized below, first as applicable to the reservoir, second as applicable to irrigation.

## RECYCLED WATER STORAGE RESERVOIR

Enforcing agencies that have been identified as having authority over the Project are the State of California Department of Water Resources Division of Safety of Dams (DSOD), Regional Board, the US Army Corps of Engineers (USACE), California Department of Fish and Game (CDFG), US Fish and Wildlife Service (USFWS), National Marine Fisheries Service (NMFS), and County of Sonoma. For each enforcing agency, applicable regulations are summarized, followed by the proposed project response to that regulation.

### A. Division of Safety of Dams (DSOD)

CA Water Code, Division 3, Part 1, Chapter 1, Section 6002, defines dam as follows:
" 'Dam' means any artificial barrier, together with appurtenant works, which does or may impound or divert water, and which either (a) is or will be 25 feet or more in height from the natural bed of the stream or watercourse at the downstream toe of the barrier, as determined by the department, or from the lowest elevation of the outside limit of the barrier, as determined by the

department, if it is not across a stream channel or watercourse, to the maximum possible water storage elevation or (b) has or will have an impounding capacity of 50 acre-feet or more." Impoundments with dams as defined above must be permitted through DSOD.

Project Response
Review by DSOD is time-consuming and typically leads to additional design elements and associated construction costs, as well as permitting fees. The District has concluded that it wants to design a pond that will be outside DSOD jurisdiction. Design of a new reservoir therefore will set the difference in elevation between the lowest outside toe of bank and the maximum water level at less than 25 feet, keeping the reservoir out of DSOD jurisprudence. See below for a discussion related to repair of the existing dam.

During identification of potential pond sites, preference will be given to sites that are outside DSOD jurisdiction. If no satisfactory sites exist that are outside jurisdiction, the District may decide to evaluate jurisdictional options. The additional costs associated with jurisdictional dam construction and permitting will need to be included in the evaluation of such sites.

CA Water Code, Division 3, Part 1, Chapter 1, Section 6225, states, "Before commencing the repair, alteration, or removal of a dam or reservoir, including the alteration or removal of a dam or reservoir so that it no longer constitutes a dam or reservoir as defined in this part, the owner shall secure the written approval of the department, except as provided in this article." An exception is allowed for emergency repairs.

Project Response
The Project may involve reconfiguration of the existing Graham's Pond. At present, the maximum operating level of Graham's Pond is determined by the overflow culvert. The topographic/bathometric survey of Graham's Pond provided by the District indicate an invert elevation of 678.0'. The plans show the toe of the embankment at an elevation of 652.0'. Current difference between the maximum operating water level and toe of dam is therefore a minimum of 26'. Under Section 6225 (above), any modifications to the reservoir would fall into DSOD jurisdiction.

## B. Regional Water Quality Control Board

Waste Discharge Requirements (WDRs) (1)
Terms that have appeared in recent WDRs include:
"A minimum freeboard, consistent with pond design, but not less than two feet, shall be maintained under normal operating conditions in any reservoir or pond containing recycled water. When extraordinary operating conditions necessitate a freeboard of less than two feet, the Discharger will document the variance in the monthly self monitoring report. The report will include an explanation of the circumstances under which the variance is required, the estimated minimum freeboard during the extraordinary period, and any permit violations occurring as a result of the variance." [Forestville Water District (FWD) WDRs]

Project Response
Reservoir will be designed with a minimum two feet freeboard. In practice, the pond will be designed with three feet freeboard at the maximum design capacity. An overflow weir

will be constructed 0.5 feet above the maximum operating water level. Head over the emergency overflow weir would not be expected to exceed 0.5 feet, assuring that freeboard would not be less than two feet, even in emergency conditions.

Waste Discharge Requirements (2)

Terms that have appeared in recent WDRs include:

"Ponds used for the storage of recycled water shall be constructed in a manner that protects groundwater. Prior to construction of any new wastewater storage ponds or use of any existing pond for storage of recycled water, the discharger shall submit to the Regional Water Board Executive Officer for review and approval, a technical report that includes design proposals and a technical evaluation that demonstrates that the pond design complies with the Water Code and Title 27 of the California Code of Regulations and is protective of ground water quality. Pond design and operation plan must include features and BMPs to protect groundwater and prevent exceedances of groundwater quality objectives." (FWD WDRs)

Project Response

See discussion below, under *Basin Plan* and Title 27.

Waste Discharge Requirements (3)

If the District is successful in designing and constructing a reservoir with adequate capacity on a new site, it will be necessary to abandon use of Graham's Pond in a manner that is acceptable to the Regional Board and can be recognized in the WDRs. Regional Board protocols for cessation of discharge of treated wastewater to a storage pond appear to be developed on a case-by-case basis. The January 2012 Regional Board meeting agenda includes a consent calendar item for "Mass Rescission" of ten WDRs. The permit holders, actions, and Regional Board statements include:

1. Wells Fargo & Fulton Processors
    a. Biosolids and wastewater were removed from treatment ponds
    b. Facilities were decommissioned
    c. Discharge of waste has ceased
    d. Rescission states that "The requirements contained in Order No. […] are no longer needed for the protection of water quality."
2. USDI Redwood Park decommissioned its WWTF
3. Other entities which simply ceased operation
    a. Regional Board determined that "the requirements contained in Order No. […] are no longer needed for the protection of water quality."
    b. "This action to rescind WDRs is not subject to the California Environmental Quality Act (CEQA) (Public Resources Code Section 21000 et seq.), pursuant to Title 14, California Code of Regulations, Sections 15060(c)(2) and 15061(b). Based on the foregoing findings and the facts underlying them, this action to rescind WDRs cannot reasonably be expected to have any potentially significant adverse effect on the environment.

Project Response

It may be necessary to develop a closure plan for Graham's Pond in a later phase of the reservoir project. Closure requirements would likely hinge on demonstration that the pond was in a condition such that return to operation as an on-stream farm pond would not reasonably be expected to have any potentially significant adverse effect on the environment. Analysis would focus on whether there were wastewater sediments in the

pond that would have potentially significant adverse effects on the environment. Depending upon the results of the analysis, it might prove necessary to remove or mitigate the pond sediments.

The District measured sludge depth at locations around the perimeter of Graham's Pond and took samples of the sludge in November 2010. Sediment depths ranged from zero to 14 inches. The sludge sample was analyzed for metals, nutrients, diesel and motor oils, volatile organic compounds, organochlorine pesticides and PCBs, semi-volatile organic compounds and dioxin. Results of the analysis were reviewed for this memorandum, for indication of presence of wastewater sludge. The lab analysis found no detectable concentrations of motor oil or organic compounds. Concentrations of copper, zinc and nitrogen compounds were generally in the low range for wastewater sludge. Concentrations of nickel and total chromium (not hexavalent chromium) were higher than typically in wastewater sludge. In addition, barium, cobalt and vanadium were detected in the sediment; these constituents are not typically analyzed in wastewater sludge, so it was not possible to evaluate the meaning of the presence of these compounds. Future analysis could include comparison with constituents in sludge taken from the District's WTF settling pond and in typical pond sediments, as well as consideration of the potential for constituents to have come from the native soils and/or from stormwater from the roads and homes in the watershed above the pond.

Mitigations to Graham's Pond required by the District's agreement with the landowners for use of the pond are outside this review of regulatory limitations, but will be included in the comparison of alternatives.

<u>Basin Plan</u> (protection of beneficial uses of groundwater)
The *Basin Plan* defines groundwater as "subsurface water in soils and geologic formations that are fully saturated all or part of the year." Water quality objectives, both numerical and descriptive, for groundwater are set forth in Section 3 of the *Basin Plan*.
The *Basin Plan* further sets forth a General Objective stating that whenever the existing water quality of a surface or groundwater is better than the Objectives for that water set forth in the *Basin Plan*, that existing quality shall be maintained. The *Basin Plan* General Objective is based on the State Water Resources Control Board Resolution No. 68-16, Statement of Policy with Respect to Maintaining High Quality Waters in California (Antidegradation Policy). The Antidegradation Policy establishes conditions that must be met "before the quality of high quality waters may be lowered by waste discharges." Those conditions, as set forth in the *Basin Plan* (page 3-2), are:
*First, the state must determine that lowering the quality of high quality waters:*
1) *Will be consistent with the maximum benefit to the people of the state,*
2) *Will not unreasonably affect present and anticipated beneficial uses of such water, and*
3) *Will not result in water quality less than that prescribed in state policies (e.g., water quality objectives in Water Quality Control Plans).*

*Second, any activities that result in discharges to high quality waters are required to:*
a) *meet waste discharge requirements that will result in the best practicable treatment or control of the discharge necessary to avoid pollution or nuisance*
b) *and maintain the highest water quality consistent with the maximum benefit to the people of the state.*

Project Response

The District will have to demonstrate that a new reservoir will not cause degradation of groundwater, either by preventing seepage or by demonstrating that potential seepage will not degrade groundwater. Construction of a new reservoir will be permitted by the Regional Board only when it has been demonstrated that the reservoir will meet the *Basin Plan* General Objective and Groundwater Objectives. Additionally, if the District selects continued use of Graham's Pond, headwaters will need to be routed around the reservoir to rectify the existing discharge violation.

There are two potential design responses to the groundwater protection regulations: (1) a pond that does not allow any seepage (two layer synthetic liner with leachate collection) or (2) a pond with a controlled rate of seepage, which has been analyzed and demonstrated to meet *Basin Plan* limitations. The liner for the second option would typically be a processed native clay, possibly with addition of bentonite, to achieve a maximum permeability of $10^{-6}$ or $10^{-7}$ cm/s. The application to the Regional Board would include characterization of groundwater quality, depths, gradients and flow; characterization of effluent quality, including all constituents regulated under the California Toxics Rule and California Department of Public Health drinking water regulations, nutrients and other constituents that affect agricultural use; analysis of projected seepage volumes and rates from the proposed reservoir, through the vadose zone to groundwater and in groundwater; projection of impacts of the reservoir to groundwater quality, immediately under the reservoir, at nearby receptors and at property lines; comparison of projected groundwater quality with *Basin Plan* Objectives.

While an anti-degradation analysis is outside the scope of the 30% conceptual design, analysis of potential pond sites will include evaluation of which approach to protecting groundwater quality might be more suited for each site. For instance, construction of a pond in a low-lying area with relatively small depth to groundwater would be more likely suited for a synthetic membrane liner system, while a site on high ground might be suitable to clay lined pond construction. The geotechnical analysis will include development of information for later use in anti-degradation analysis, such as groundwater elevations and a discussion of soil permeabilities. Cost estimates will include an estimate of the comparative costs associated with a clay liner versus synthetic membrane liner system, including costs associated with securing the appropriate permits through the Regional Board.

Title 27

The State Water Resources Control Board (SWRCB) issued a decision (2009 "Lodi Decision"), that storage ponds must be part of the treatment process in order to be exempted from Title 27 liquid waste management regulations. Entities proposing to construct effluent storage reservoirs were required to either (a) meet Title 27 construction requirements, (b) demonstrate that the design provides equivalent protections to Title 27, or (c) obtain an exemption from Title 27.

The Lodi Decision was amended in February 2012 through Order WQ 2012-0001. The revised order states the Title 27 municipal wastewater treatment plant exemption does apply to post-treatment storage facilities if the facilities (1) are used to store treated municipal wastewater prior to ultimate disposal or reuse, (2) do not receive any other wastes other than authorized on-site stormwater flows, and (3) are under the control of the municipal treatment plant.

Project Response
No response needed, as the project is exempt from Title 27.

Title 22, Code of California Regulations
Article 4, Section 60310 prohibits "irrigation with, or impoundment of, disinfected secondary-2.2 or disinfected secondary-23 recycled water shall take place within 100 feet of any domestic water supply well."

Project Response
Site selection will comply with this set-back requirement.

Section 401 Water Quality Certification
Construction of the reservoir may have some impact to wetlands and/or waters of the US and waters of the state requiring a permit from the Regional Board and USACE (further described in item C. below), respectively. Modification of Graham's Pond would impact wetlands and result in realignment of creek channel (headwaters above the dam), also requiring a permit from the Regional Board and USACE. Section 401 of the Clean Water Act requires that the State certify the USACE permit. Title 23, Chapter 28 of the California Code of Regulations grants authority for Section 401 Water Quality Certification to the Regional Boards.

The Regional Board requires mitigation for impacts to waters of the State. Mitigation can be congruent with USACE mitigation requirements or can exceed those requirements. Additionally, the Regional Board can extend its authority to include waters outside of the USACE jurisdiction under the Porter-Cologne Water Quality Act. Typically, when a Section 401 Water Quality Certification is issued, any such additional jurisdiction and mitigation requirements are included.

Project Response
To the extent possible, project elements will be located to avoid these resources to reduce impacts and mitigation expenses. The Environmental Information Document (EID) will include discussion regarding the need for Section 401 Water Quality Certification and potential mitigation measures.

## C. California Air Resources Board

The California Air Resources board (CARB) regulates construction activities within areas of ultramafic rock[1], to protect air quality and human health from potential for airborne asbestos. Regulations are contained in two Asbestos Airborne Toxic Control Measures (ATCM). The following elements of the regulations would be pertinent to the Project if construction is planned within mapped areas of serpentine, which is a type of ultramafic rock:

1. Under the "Asbestos ATCM for Construction, Grading, Quarrying and Surface Mining Operations" (2002), if a proposed project is in an area that is designated on State Geologic

[1] Ultramafic rock is igneous rock with ferro-magnesium materials. Serpentine is a type of ultramafic rock. Serpentine may or may not contain asbestos.

Maps as "ultramafic rock unit or ultrabasic rock unit", or if such rock is found in a disturbed area during construction, certain procedures must be adhered to, as follows:

a. If the area to be disturbed is less than one acre, the following limitations apply:
    i. Construction vehicle speed at the work site must be limited to fifteen (15) miles per hour or less
    ii. Prior to any ground disturbance, sufficient water must be applied to the area to be disturbed to prevent visible emissions from crossing the property line
    iii. Areas to be graded or excavated must be kept adequately wetted to prevent visible emissions from crossing the property line;
    iv. Storage piles must be kept adequately wetted, treated with a chemical dust suppressant, or covered when material is not being added to or removed from the pile;
    v. Equipment must be washed down before moving from the property onto a paved public road; and
    vi. Visible track-out on the paved public road must be cleaned using wet sweeping or a HEPA filter equipped vacuum device within twenty-four (24) hours.
b. If the area to be disturbed is greater than one acre, an Asbestos Dust Mitigation Plan must be developed, submitted to and approved by the local air quality district, implemented and maintained throughout the duration of the construction project.
c. The Asbestos Dust Mitigation Plan must address the following:
    i. Track-out prevention and control measures
    ii. Keeping active storage piles adequately wetted or covered with tarps
    iii. Control for disturbed surface areas and storage piles that will remain inactive for more than seven (7) days
    iv. Control for traffic on on-site unpaved roads, parking lots, and staging areas
    v. Control for earthmoving activities
    vi. Control for Off-Site Transport
    vii. Post Construction Stabilization of Disturbed Areas
    viii. Air Monitoring for Asbestos (If Required by the Air Pollution Control Officer.
d. An exemption may be granted if it is demonstrated that the earth in the area to be disturbed contains no detectable asbestos.
e. Individuals engaged in construction activities on residential property they own or occupy are exempt from the CARB requirements.

2. Materials greater than 0.25% asbestos may not be used in road surfacing applications, except "covering composed of asphalt concrete or portland cement concrete". Because of this limitation, such materials are not generally accepted by quarries.
3. Waste that contains asbestos at greater than 1% is categorized as hazardous waste in Title 22, Section 66261.24. This concentration limit applies "only if the substances are in a friable, powdered or finely divided state."
4. Regulations do not prohibit use of serpentine materials in construction as long as they do not remain exposed upon project completion.

The nearest landfill, Redwood Landfill in Novato, is able to use materials with up to 1% friable asbestos or up to 5% non-friable asbestos for alternative daily cover, as long as there is not "too much" rock in the mix.

Project Response

The Geologic Map of the Santa Rosa Quadrangle (Wagner, D.L. and Bortugno, E.J. (Reprinted 1999) shows a band of ultramafic rock running through the project area. If a proposed pond site would involve construction within the area designated as ultramafic, or an area with apparent ultramafic rocks, representative samples of the rock will be tested for asbestos. If asbestos is found, a project response will be developed to minimize disturbance of areas with ultramafic rock and to meet applicable regulations for air quality protection.

D. CalRecycle

Title 22, Sections 66261.2, 66261.24, 66268.29

Title 22 states that asbestos-containing waste is defined as hazardous if it contains friable asbestos at a concentration of 1% or greater. No limit is given for non-friable asbestos. "Waste" is defined as any discarded material in any form. Hazardous waste may not be disposed at a Class III landfill.

Project Response

As stated above, samples of soil and rock from proposed pond sites in mapped ultramafic rock areas will be tested for asbestos content. Ultramafic rock may be used on site for construction, as long as it is not exposed at project completion. Caltrans, for example, requires 3" of non-ultramafic earth cover over construction using ultramafic materials. However, any pond alternative which required off-hauling of earthen materials, and for which preliminary testing indicated asbestos concentrations of 1% or greater, would be viewed as very risky in terms of potential for need for hazardous waste disposal. Even though asbestos that is in rock would not be defined as friable, excavation and crushing of rock would risk producing friable asbestos. Caltrans, for example, treats all earthen material containing naturally occurring asbestos (NOA) at equal to or greater than one percent as a hazardous waste. The evaluation of potential pond sites will include assessment of the risk of generation of hazardous waste from excavation of ultramafic material.

**E. US Army Corps of Engineers (including US Fish & Wildlife Service, National Marine Fisheries and Section 106 of the National Historic Preservation Act)**

Section 404 of the Clean Water Act

Under Section 404 of the Clean Water Act, the USACE has regulatory authority over discharge of dredged or fill material into waters of the United States. 33 CFR part 328 defines water of the United States as:

(a) The term waters of the United States means

(1) All waters which are currently used, or were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters which are subject to the ebb and flow of the tide;

(2) All interstate waters including interstate wetlands;

(3) All other waters such as intrastate lakes, rivers, streams (including intermittent streams), mudflats, sandflats, wetlands, sloughs, prairie potholes, wet meadows, playa lakes, or natural ponds, the use, degradation or destruction of which could affect interstate or foreign commerce including any such waters:

(i) Which are or could be used by interstate or foreign travelers for recreational or other purposes; or
(ii) From which fish or shellfish are or could be taken and sold in interstate or foreign commerce; or
(iii) Which are used or could be used for industrial purpose by industries in interstate commerce;

(4) All impoundments of waters otherwise defined as waters of the United States under the definition;
(5) Tributaries of waters identified in paragraphs (a) (1) through (4) of this section;
(6) The territorial seas;
(7) Wetlands adjacent to waters (other than waters that are themselves wetlands) identified in paragraphs (a) (1) through (6) of this section.
(8) Waters of the United States do not include prior converted cropland. Notwithstanding the determination of an area's status as prior converted cropland by any other Federal agency, for the purposes of the Clean Water Act, the final authority regarding Clean Water Act jurisdiction remains with EPA.

Project Response

Based on the wetland delineation of the Loades site conducted in 2003, there are significant areas of waters of the United States (wetland and surface waters). Construction of a new reservoir or modification of the existing Graham's Pond may result in placement of fill into those waters. There are two permitting paths under Section 404 of the Clean Water Act: a Nationwide Permit and an Individual Permit. The appropriate Nationwide Permit 18 regulating placement of fill in wetlands is limited to fill of up to 1/10 acre of wetland fill. Considering the known extent of wetlands on the project site, an Individual Permit will likely be required. Coverage for any impacts to streams below the ordinary high water mark would also need to be covered by the Individual Permit. The USACE requires that impacts to waters be mitigated, generally at some ratio exceeding 1:1.

As part of the Section 404 permitting process, the USACE will consult with USFWS and NMFS under Section 7 of the Endangered Species Act. Based on the biological assessment conducted for the Draft Environmental Impact Report: Camp Meeker Wastewater Reclamation Project (October 2001 EIR), the potential exists for both terrestrial and aquatic listed species to be present on or adjacent to the project area. If USFWS and/or NMFS believes there is a potential for the project to impact a protected species, incidental take authorization would need to be included in the USACE permit. Incidental take authorization would be accompanied by minimization and avoidance measures that would restrict construction activities and possibly by mitigation requirements.

USACE will also consult with the State Historic Preservation Office/Tribal Historic Preservation Officer (SHPO/THPO) for compliance with Section 106 of the National Historic Preservation Act. This consultation will involve local Indian Tribes who may have an interest in the project area.

To the extent possible, project elements will be located to avoid these resources to reduce impacts and mitigation expenses. The EID will include (1) a preliminary wetland delineation (This delineation will not be verified by the USACE. Timing requires the delineation be conducted in January/February when plant species have not flowered and

may require additional delineating in later spring) and (2) a habitat assessment to determine the likelihood of listed species presence and identification of the need for protocol-level studies. The EID will also include discussion regarding the need for Section 404 and 401 permitting, incidental take authorization and potential mitigation measures. The EID will include a cultural resources investigation, including outreach to local Indian Tribes, to determine potential impacts to historical or archaeological resources.

## F. California Department of Fish and Game

Section 1602 of the Fish and Game Code

Fish and Game Code (Section 1602) requires CDFG notification for any proposed activity that may substantially modify a river, stream, or lake, generally known as a streambed alteration agreement.

> 1602. (a) An entity may not substantially divert or obstruct the natural flow of, or substantially change or use any material from the bed, channel, or bank of, any river, stream, or lake, or deposit or dispose of debris, waste, or other material containing crumbled, flaked, or ground pavement where it may pass into any river, stream, or lake.

Project Response

Continued utilization of Graham's Pond would result in the need for a Section 1602 permit for the realignment of the headwaters and potentially for improvements to the dam since it is an on-stream reservoir. Any other project elements that would impact streams would similarly require a Section 1602 permit. CDFG requires mitigation for impacts to resources under its jurisdiction, generally in excess of 1:1. To the extent possible, project elements will be located to avoid these resources to reduce impacts and mitigation expenses. The EID will include identification of CDFG jurisdictional resources and will include discussion regarding the need for Section 1602 permitting, including potential mitigation measures.

Section 2080 of the Fish and Game Code

Section 2080 of the Fish and Game Code prohibits "take" of any species that the commission determines to be an endangered species or a threatened species. Sections 2081(b) and (c) of the California Endangered Species Act allow CDFG to issue an incidental take permit for State listed threatened and endangered species only if specific criteria are met as defined by Title 14 CCR Section 783.4:

> (a) Issuance criteria. If an application is submitted in accordance with section 783.2 and section 783.3, the Director shall decide whether or not an incidental take permit should be issued. A permit may only be issued if the Director finds that:
>
> > (1) The take authorized by the permit will be incidental to an otherwise lawful activity.
> > (2) The applicant will minimize and fully mitigate the impacts of the take authorized under the permit. The measures required to meet this obligation shall be roughly proportional in extent to the impact of the authorized taking on the species. Where various measures are available to meet this obligation, the measures required shall maintain the applicant's objectives to the greatest extent possible. All required measures shall be capable of

successful implementation. For purposes of this section only, impacts of taking include all impacts on the species that result from any act that would cause the proposed taking.
(3) The permit will be consistent with any regulations adopted pursuant to Fish and Game Code Sections 2112 and 2114.
(4) The applicant has ensured adequate funding to implement the measures required under the permit to minimize and fully mitigate the impacts of the taking, and to monitor compliance with, and the effectiveness of, the measures.

Project Response
Based on the biological assessment conducted for the October 2001 EIR, the potential exists for both terrestrial and aquatic listed species to be present on or adjacent to the project area. As indicated above, impacts to protected species require mitigation. CDFG also typically requires minimization and avoidance measures that would restrict construction activities.

To the extent possible, project elements will be located to avoid protected species to reduce impacts and mitigation expenses. The EID will include identification of CDFG protected species and will include discussion regarding the need for Section 2080 permitting, including potential mitigation measures.

### G. County of Sonoma and private right-of-way and/or easement agreements

Depending on the location of the reservoir, the need for County of Sonoma encroachment permits and/or easements may be required. Easements or property acquisition in fee is required before construction of public works on private property. Encroachment permits are required by the County of Sonoma before construction of new utilities within the public right of way.

Project Response
The Engineering Report will identify where easements and/or encroachment permits would be required.

## RECYCLED WATER IRRIGATION

### A. Regional Water Quality Control Board

Waste Discharge Requirements (1)
Terms that have appeared in recent WDRs include:
"All new recycled water use sites shall include a 100-foot setback to all surface waters or provide written documentation of appropriate best management practices that will be implemented in order to prevent or minimize the potential for runoff discharging to surface water" (FWD WDRs)

Project Response
The District will assert that irrigation along the ephemeral stream at Loades is a continuing use and is therefore exempt from this requirement.

New irrigation areas will either be configured to include a 100-foot setback from surface waters or developed in a manner to prevent or minimize the potential for runoff discharging to surface water.

*Comment*: Regional Board staff has not provided basis for this requirement since queried in September 2011.

Waste Discharge Requirements (2)

Terms that have appeared in recent WDRs include:

"This Order also includes a process for Regional Water Board approval of new recycled water use sites that would receive discharges to land. This approval process will include compliance with CEQA as necessary. The approval process requires demonstration that a CEQA analysis has been conducted for any proposed recycled water use project. The approval process also requires the Discharger to submit technical information necessary to demonstrate that any proposed recycled water use areas will be irrigated using the most stringent of the hydraulic and nutrient agronomic rate and include best management practices that are protective of surface and ground water quality." (FWD WDRs)

Project Response

The hydraulic and nutrient loading rate analysis will be addressed in a later phase of the project. Similarly, the CEQA document would address impacts related to hydraulic and nutrient loading rates in a later phase.

Title 22

The District intends to obtain a permit for "Disinfected secondary-2.2 recycled water," which is defined as "recycled water that has been oxidized and disinfected so that the median concentration of total coliform bacteria in the disinfected effluent does not exceed a most probable number (MPN) of 2.2 per 100 milliliters utilizing the bacteriological results of the last seven days for which analyses have been completed, and the number of total coliform bacteria does not exceed an MPN of 23 per 100 milliliters in more than one sample in any 30 day period." Allowable irrigation with Secondary-2.2 recycled water includes food crops where the edible portion is produced above ground and not contacted by the recycled water, pasture for animals producing milk for human consumption, and other uses with lower potential for human exposure.

Project Response

Any new irrigation areas will be screened for possible issues with these requirements. The first priority for irrigation will be lands with pasture and/or fodder crops. If the irrigation area is to include vineyards, spray irrigation will not be allowable.

Title 22

No irrigation with, or impoundment of, disinfected secondary-2.2 or disinfected secondary-23 recycled water shall take place within 100 feet of any domestic water supply well.

Project Response

Any new irrigation areas will be screened for possible issues with this requirement.

Recycled Water Policy

In 2009, the State Water Resources Control Board adopted the Recycled Water Policy (RWP). The RWP was developed to increase the use of recycled water and conservation of potable water in California. The RWP addresses landscape irrigation and groundwater recharge. The RWP does not address agricultural irrigation projects.

The RWP states an intent that every groundwater basin/sub-basin in California have a consistent salt/nutrient management plan.

Project Response

Because there is neither landscape irrigation nor groundwater recharge associated with the current project, the RWP would not apply to the project. In respect to Salt/Nutrient Plans, State mapping of groundwater basins (Department of Water Resources Bulletin 118, "California's Groundwater") indicates that the project area is not within a recognized groundwater basin. In any event, development of a Salt/Nutrient Plan is not the responsibility of any one particular project, but of all local water and wastewater entities, together with local salt/nutrient contributing stakeholders.

For the reasons given in the previous paragraph, no project response is needed.

## B. County and private right-of-way and/or easement agreements

Depending on the location of the reservoir, the need for County of Sonoma encroachment permits and/or easements may be required. Easements or property acquisition in fee is required before construction of public works on private property. Encroachment permits are required by the County of Sonoma before construction of new utilities within the public right of way.

Project Response

The Engineering Report will identify where easements and/or encroachment permits would be required.

## Summary List of the Requirements of Governmental Authorities Having Jurisdiction to Approve Portions of the Project

| Enforcing Agency | Requirements |
|---|---|
| Regional Water Quality Control Board | Submittal of Report of Waste Discharge with supporting Engineering Report to address Basin Plan requirements for protection of beneficial uses of groundwater, Title 22 requirements, and change from NPDES permit to WDRs.<br>Section 401 Water Quality Certification for impacts to wetlands and abandonment of Graham's Pond<br>Submittal of petition in compliance with Water Code Section 1700 requirements for ceasing surface water discharge |
| State Water Resources Control Board | Compliance with the Statewide General Construction Stormwater Discharge Permit, by filing a Notice of Intent with the State Water Resources Control Board (SWRCB) in compliance with the Construction General Permit, and by preparing and implementing a stormwater pollution prevention plan (SWPPP) and an erosion control plan. |
| California Department of Public Health | Submittal of copy of ROWD for review for compliance with Title 22 requirements, in particular well set-back limits. |
| California Air Resources Board | Compliance with requirements of the Asbestos Airborne Toxic Control Measures listed in the "Asbestos ATCM for Construction, Grading, Quarrying and Surface Mining Operations" for construction within areas with asbestos-bearing soils.<br>Submittal an Asbestos Dust Mitigation Plan to the Northern Sonoma County Air Pollution Control District if the area of construction within asbestos-bearing soils exceeds one acre. |
| County of Sonoma Permit and Resource Management Department | Submittal of a zoning permit application to the County of Sonoma.<br>Compliance with a General Plan Consistency Determination during the CEQA review process.<br>Application for a less than 3 acre Timber Conversion Exemption permit for tree removal.<br>Application for a Roadway Encroachment Permit.<br>Application for a Grading Permit. The grading permit generally includes requirements contained in the *Stormwater Low Impact Development (LID) Technical Design Manual* to reduce construction related and post-construction water quality impacts. |
| US Army Corps of Engineers | Submittal of a Section 404 Nationwide or Individual Permit application for impacts to wetlands and other waters will be required. |
| US Fish and Wildlife Service | Compliance with Section 7 of the Federal Endangered Species Act. Consultation may be required for federally listed plant and animal species including the California red-legged frog (*Rana draytonii*). Pre-construction nesting bird surveys will also occur. |
| State Historic Preservation Office | Compliance with Section 106 of the National Historic Preservation Act will be required associated with the USACE permit. |
| California Department of Fish and Game | Compliance with the California Endangered Species Act. Section 2081 Incidental Take Permit may be required for state listed plant and animal species. Protocol-level surveys will be conducted for the following: Red tree vole (*Arborimus longicaudus*) and northern spotted owl (*Strix occidentalis caurina*). Pre-construction nesting bird surveys will also occur.<br>Seasonally appropriate sensitive plant surveys in accordance with CDFG's 2009 guidelines will be conducted prior to soils, hydrology, or plant altering activities for the following: seaside tarplant (*Hemizonia congesta ssp. congesta*), short-leaved evax (*Hesperevax sparsiflora var. brevifolia*), Baker's manzanita (*Arctostaphylos bakeri ssp. bakeri*), and fragrant fritillary (*Fritillaria liliacea*).<br>Submittal of a Section 1602 Lake and Streambed Alteration Agreement would be required for abandonment of Graham's Pond and for improvements to streams. |
| California Department of Forestry | Application for a Less than 3 acre Timber Conversion Exemption permit for tree removal. |

# APPENDIX C

# GEOTECHNICAL REPORT

Prepared for **Brelje & Race Consulting Engineers**

# GEOTECHNICAL INVESTIGATION
# 30% CONCEPTUAL DESIGN PHASE
# OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
# SITE 3A & SITE 3C-2
# OCCIDENTAL, CALIFORNIA

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February 13, 2013

Project No.: 01202/GEO2

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Brelje & Race Consulting Engineers
475 Aviation Blvd., Suite 120
Santa Rosa, CA 95403

Attention: Mr. Richard Ingram
Email: ingram@brce.com

**Subject: Geotechnical Investigation 30% Conceptual Design Phase**
**OCSD Wastewater Storage and Reclamation Project**
**Site 3A & Site 3C-2, Occidental, California**

Dear Mr. Ingram:

We are pleased to submit our geotechnical investigation report for the above referenced project. This report provides the results of our field investigation, laboratory testing, and engineering analysis. Recommendations for the earthwork aspects of the project and mitigation of geotechnical and geologic concerns are included.

It should be noted that the conclusions and recommendations presented in this report are based on limited subsurface investigation data and as a result, variations between anticipated and actual soil and bedrock conditions may be found during construction. We should therefore be engaged to observe and document the earthwork construction, during which time we can modify our recommendations if different site conditions are encountered. In addition, design plans should be reviewed by this office prior to their issuance. These additional services are further discussed in the report. We appreciate the opportunity of providing geotechnical engineering services for you on this project. Please contact us as needed.

Sincerely,

**M2 Consultants, Inc.**

Yogesh Prashar, G.E.
Principal Geotechnical Engineer

Alastair Johns, P.G.
Principal Geologist
Operations Manager


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# *Executive Summary*

*This report is submitted by M2 Consultants (M2C) to Brelje & Race Consulting Engineers (BRCE) in support of a 30% design submittal to the project owner, the Occidental County Sanitation District (OCSD). It presents the results of geotechnical investigations of two proposed 12.5 million gallon wastewater reservoir sites, which are referred to as Site 3A and Site 3C-2. They are located just outside the town of Occidental, West Sonoma County, California (Plate 1).*

*The purpose of this report is to (1) identify geological and geotechnical constraints and advantages to development and (2) establish criteria for the design and construction of proposed earthen construction storage reservoirs at the sites. Work performed included office review of available existing geologic publications, geological field mapping, subsurface geotechnical explorations (test pits, soil and rock borings, CPTs and a seismic refraction survey), laboratory analyses and stability evaluations.*

*The site locations are presented on Plate 2. The Site 3A reservoir site is located on the Loades North property immediately north of the existing 8.5 MG 'Graham's Pond' reservoir. The Site 3C-2 reservoir site is located on the southern portion of the Loades South property parcel near Facendini Lane. M2C previously submitted a report entitled 'Geotechnical Investigation, 30% Conceptual Design Phase OCSD Wastewater Storage and Reclamation Project, Occidental, California' dated April 4, 2012 which included a design level geotechnical investigation for the proposed Site 3A reservoir area that is addressed and superseded by this report.*

*Detailed investigation initially focused on Site 3A. However, Site 3A was judged inadequate for cost-effective construction due factors including high groundwater, weak expansive soils, liquefaction potential, inadequate slope and earthquake stability and asbestiform materials in serpentinite bedrock and overlying soils within the excavation footprint. As a result the investigation was broadened, which led to the detailed investigation of Site 3C-2.*

*The results of the investigation of Site 3C-2 indicate favorable conditions for construction and long-term usage. Site 3C-2 is underlain by rippable, highly weathered, weak sandstone conglomerate to 24 to 41 feet below the existing ground surface, which is underlain by more competent but still weak, weathered and variable graywacke sandstone and shale sedimentary sequences. These materials can be used for embankment fill construction. Excess material could be exported for use as fill on other sites. Stability analyses of the approximately 45 foot deep excavation below existing grade necessary to establish the reservoir show that the interior and exterior slopes will be stable in construction, in long term static loading and during rapid drawdown conditions (sudden drainage of the reservoir). Seismic analysis, based on the maximum credible earthquake (MCE) of magnitude 7.9 from the San Andreas Fault (which is approximately 12.16 km (7.6 miles) away and is the controlling design seismic source for the site) indicate an adequate factor of safety. No known active faults cross the site so fault rupture is not anticipated.*

*Serpentinite bedrock containing asbestiform minerals is located immediately to the southwest of the proposed 3C-2 reservoir footprint and extends to the depths explored. No evidence of the occurrence of serpentinite was observed in the excavation area. As long as the area of serpentinite remains undisturbed, it does not affect the construction feasibility of the 3C-2 reservoir.*

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# 1.0 INTRODUCTION

This report presents the results of a geological and geotechnical investigation for two proposed reclaimed water storage reservoir sites, termed Site 3A and Site 3C-2, to be located in near the town of Occidental, West Sonoma County, California (Plate 1, Site Vicinity Map). The sites are located about half a mile to the east of the town of Occidental and bounded by Graton Road to the north, Facendini Lane to the east and Occidental Road to the southwest (Plate 2, Site Parcel Plan). The Proposed Pond Site 3A reservoir is located within the western portion of the 40.21 acre Loades (north) Parcel #074-080-063. The Proposed Pond Site 3C-2 reservoir is located within the southern portion of the 63 acre Loades (south) Parcel #074-070-010.

*Figure 1: View looking north over Graham's Pond in-take toward existing dam*

## 1.1 PROJECT DESCRIPTION

The project consists of the improvement or replacement of an existing reclaimed water storage pond named 'Graham's Pond', (Figure 1 and Plate 2). In addition to the two parcels described above, two other parcels were selected for initial consideration:

1. Briggs Parcel (APN 074-080-005)
2. Colella Parcel (APN 074-080-057)

The current Graham's Pond reservoir and dam is on private property owned by the Loades family and is operated by OCSD under an easement from the property owner. Site 3A is located adjacent to the north end of Graham's Pond.

Previous work identified geologic hazards and construction considerations for the four parcels that are summarized in Table 1 – Geologic Hazards & Construction Considerations (Appendix E). Geologic site maps for the four parcels are included as Plates 13, 14 & 15 and the Geologic Site Maps Plan as Plate 12. After review of the initial feasibility level information, the OCSD selected Site 3A and then later, Site 3C-2, for detailed geotechnical investigation.

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## 1.2 WORK PERFORMED

### 1.2.1 Site 3A

In addition to the screening work described above, the following work was performed for the Site 3A geotechnical design:

- BACKGROUND FILE REVIEW of available geotechnical reports, maps and other data regarding the geology, tectonics, and seismicity of the entire site vicinity.
- SITE RECONNAISSANCE AND GEOLOGIC MAPPING of the site.
- Six TEST PITS to initially identify and characterize bedrock characteristics.
- SUBSURFACE FIELD INVESTIGATION consisting of 4 soil borings/rock cores and 6 CPTs to evaluate subsurface conditions and provide data for geotechnical design analyses.
- GEOTECHNICAL LABORATORY TESTING of subsurface soil samples and rock cores from the field investigation for use in design analysis.
- Development of SITE SPECIFIC STRONG GROUND-MOTIONS in accordance with regulatory and International Building Code (IBC) requirements.
- GEOTECHNICAL DESIGN ANALYSIS for embankments, foundations and appurtenances under static and dynamic conditions and evaluation of liquefaction and other permanent ground deformation hazards to the proposed development.
- Preparation of a draft GEOTECHNICAL REPORT, providing conclusions and recommendations for the design and construction for the preferred design alternative, for review and comment. Preparation of a final report, taking into account review comments.

*The Site 3A final report dated April 4, 2012, concluded that based on the results of the analysis the configuration as proposed did not successfully meet parameters that are necessary to provide for a safe and reliable embankment structure.*

### 1.2.2 Site 3C-2

*The following work was performed for Site 3C-2:*

- DATA REVIEW for Site 3C-2 of new, pertinent and available information including documents prepared by BRCE (grading plans, and cross sections, geologic maps).
- GEOLOGIC EVALUATION including review of existing available reports and data, field mapping, photo geologic study of available USGS aerial photos, identification of geologic hazards, and documentation of structural bedrock characteristics.
- SUBSURFACE FIELD EXPLORATION consisting of the following:

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- TEST PITS - comprising 12 additional test pits to further evaluate formation lateral boundaries, with particular attention to the delineation of the serpentinite.
- SEISMIC REFRACTION data along two profiles to support the evaluation of depth, configuration, and excavation characteristics (rippability) of the materials underlying the planned reservoir.
- 8 BORINGS advanced in and around the proposed pond area up to 60 feet depth. Drum disposal of drill cuttings and drilling fluid.

- GEOTECHNICAL LABORATORY TESTING of subsurface soil samples and rock cores for use in design analysis.
- GROUND MOTION ANALYSIS: Development of site specific strong ground-motions in accordance with the regulatory or IBC requirements.
- STATIC & DYNAMIC ANALYSIS OF EMBANKMENTS, FOUNDATIONS & PIPELINES including development of material parameters using Hoek-Brown failure envelope, for use in stability evaluations.
- ROCK SLOPE STABILITY using limit equilibrium embankment stability for the interior and exterior slopes.
- PIPELINE AND APPURTENANCES ANALYSIS AND DESIGN: Analysis, design and construction recommendations for the proposed 6 inch inlet / outlet pipeline.
- ENGINEERING REPORT - Preparation of this revised draft geotechnical report (originally submitted primarily for the 30% design for the Site 3A-2 Pond).

Environmental services such as evaluation and chemical analysis of the soil and groundwater for hazardous materials were not included in our scope of services.

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# 2.0 GEOLOGY

## 2.1 GEOLOGIC SETTING

### 2.1.1 Regional Geology

Sonoma County is located within the Coast Range Geomorphic Province, a more or less discontinuous series of northwest trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. During the last 25 million years the geologic and geomorphic structures were primarily created and dominated by faulting, which continues to the present day.

### 2.1.2 Regional Faulting

The San Andreas Fault system is a right-lateral, strike-slip fault zone that extends mostly on land from the Gulf of California in Mexico, to Cape Mendocino on the Humboldt County coast in northern California. The San Andrea fault itself defines the boundary between the North American Plate to the east and the Pacific Plate to the west. The Pacific Plate has been moving 1-2 inches a year to the north, relative to the North American Plate for the last 25 million years. The San Andreas Fault is located approximately 12.16 km (7.6 miles) to the west of the site and is the design fault for this project.

### 2.1.3 San Francisco Bay Area Faulting

Within the region encompassing West Sonoma County, movement across this plate boundary is primarily distributed across the San Andreas Fault and the Rogers Creek fault which is located approximately 17.12 km (10.7 miles) to the ENE. These and a number of other major faults including the Hayward and Calaveras faults are shown on Plate 6 - Regional Fault Map. Together, these faults are referred to as the San Andreas Fault System (SAF). The general N40º to 45ºW trend of the faults within this system is responsible for the strong northwest-southeast structural grain of geologic and geomorphic features in Sonoma County and the entire Coast Range geomorphic province, including the San Francisco Bay Area to the south.

For most of the length of the San Andreas Fault, basement rock on the west side is generally of Cretaceous Age (about 65 to 140 million years old) and is granitic rock of the 'Salinian block' unit. To the east of the fault, basement bedrock consists of a chaotic mixture of highly deformed marine sedimentary, submarine, volcanic, and metamorphic rocks of the Franciscan Complex.

### 2.1.4 Quaternary Sediments

The inland valleys in West Sonoma County are generally filled with unconsolidated to semi-consolidated deposits of Quaternary Age (about 1.6 million years to present). These are continental

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deposits (alluvium and colluvium) consisting of unconsolidated to consolidated sand, silt, clay, and gravel mixtures.

### 2.1.5 Great Valley Complex[1]

The Great Valley complex is composed of at least five terranes of mainly sedimentary sequence rocks, of which the Healdsburg terrane, the Del Puerto terrane, and the Coast Range ophiolite crop out in the study area and are summarized below. Although the origin of the Great Valley Complex remains controversial, it is thought that it formed in the fore-arc of an oceanic island arc outboard of the North American continental margin, and the island arc and the fore arc-related ophiolite were subsequently accreted to North America during the late Kimmeridgian Nevadan Orogeny (approximately 151-153 Ma).

#### 2.1.5.1 Healdsburg Terrain

*Map symbol KJgv in the study area on Plate 4 (Sandstone, shale, and conglomerate – Early Cretaceous and Late Jurassic).*

The type area is near Healdsburg, where as much as 3,000 m of Lower Cretaceous conglomerate depositionally overlie Upper Jurassic shale over keratophyre, basalt, gabbro, and peridotite of the Coast Range ophiolite. The Healdsburg terrane is distinguished from coeval Great Valley sequence rocks by its abundant conglomerate containing clasts mostly of light-colored (often pink) rhyolite porphyry and rhyolitic welded ash-flow tuff, plus minor quartzite (quartz arenite) pebbles.

#### 2.1.5.2 Del Puerto Terrane

*Map symbol Jk and Jsv in the study area on Plate 4 (respectively Knoxville Formation – Late Jurassic; Keratophyre and quartz keratophyre tuff – Late Jurassic).*

The type area is in the eastern Diablo Range where the basal part of the stratigraphic sequence in the terrane is composed of dismembered ophiolite and a thick accumulation of silicic volcanic rocks (keratophyre and quartz keratophyre), overlain by silicic tuff and tuffaceous sandstone of the Upper Jurassic Lotta Creek Formation and Upper Jurassic to Lower Cretaceous turbidites. These Del Puerto terrane rocks are structurally interleaved with rocks of the Franciscan Complex and Healdsburg terrane and are more than 100 km from the main body of Del Puerto terrane.

[1] Blake, M.C., Jr., Graymer, R.W., and Stamski, R.E., 2002, Geologic map and map database of western Sonoma, northernmost Marin, and southernmost Mendocino, counties, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-2402.

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#### 2.1.5.3 Coast Range ophiolite

*Map symbol sp & spm in the study area on Plate 4 (respectively Serpentinite – Late and Middle Jurassic; Serpentinite matrix mélange – Late and Middle Jurassic).*

The ophiolitic rocks formed between about 164 and 173 Ma. and make up the basal part of all the Great Valley complex terranes in the study area and include most of the rock types that make up a typical ophiolite suite, including serpentinite, pyroxenite, gabbro, diabase, and massive and pillowed basalt. The Serpentinite is structurally interleaved with Franciscan Complex mélange or contains high-grade metamorphic blocks (previously mapped as part of the Franciscan Complex).

### 2.1.6 Franciscan Complex

The Franciscan Complex are generally considered to be of Jurassic and Cretaceous Age (about 65 to 205 million years old) and are composed of many terranes, plus mélange (a chaotic mix of rocks derived from multiple terranes). The Franciscan rocks have typically been extensively folded and faulted largely because of movement along the San Andreas Fault system over the last 25 million years and consequently includes a wide variety of rocks including deep-sea radiolarian cherts, sandstones, limestone's, shales, serpentinites, basalts, volcanic tuffs, altered mafic volcanic rocks (greenstones), high-pressure metamorphic rocks and a wide array of other rocks, all of them faulted and mixed in a seemingly chaotic manner and commonly metamorphosed to varying grades. The Franciscan Formation forms the major component of the Pacific Coast Ranges of California. The subduction emplacement of the Franciscan Complex means that Franciscan Complex terranes were originally all structurally under the Great Valley complex. This juxtaposition allowed some admixture of materials of the two terrane complexes. Specifically, serpentinite from the base of the Coast Range ophiolite (Great Valley complex) is found entrained in the Franciscan Complex mélange, whereas high-grade metamorphic blocks (Franciscan Complex) are found entrained in serpentinite matrix mélange that is associated with the Coast Range Ophiolite.

## 2.2 LOCAL GEOLOGY

### 2.2.1 Quaternary Units

The alluvial deposits (map symbol Qa) shown on (Plates 4, 13, 14, 15), are generally undivided sand, gravel, silt and clay mixtures, and in the site area occur as active stream channel units and small alluvial fans. These deposits underlie most of the Site 3A valley area and the proposed new Site 3A reservoir location, and include the valley area on either side of the Graton Road in the portions of the Briggs, Colella and Loades North parcels in the site area.

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### 2.2.2 Bedrock Units

Per the Geologic Map of the Camp Meeker 7.5' Quadrangle (Delatorre et al, 2009), a portion of which is included as the Local Geologic Map (Plate 4); field mapping by M2C (geologic map Plates 13, 14 & 15); Google Earth images, and other topographic and geologic maps (see references), the vicinity of Sites 3A and 3C-2 is located where the youngest lithified geologic bedrock strata in the area, the Wilson Grove Formation (map symbol Twg) marine sandstone (late Pliocene to late Miocene ~3.5 – 7.5 Ma) occurs on the higher ground surrounding sites and has been irregularly eroded revealing Cretaceous (65-145 MA) and Jurassic (~145 – 200 MA) basement formations comprising (inactive) fault bounded serpentinite matrix mélange, Knoxville formation shale, Keratophyre tuff (extrusive volcanics) and Franciscan Complex mélange.

*Figure 2: Highly variable, weak, friable to plastic, pervasively sheared serpentinite matrix with pods of entrained slightly less weathered but weak serpentinite within the Dutch Bill Creek diversion creek cut and typical of the site area*

Serpentinite outcrops on the northeast side of a northwesterly draining valley containing the northeast half of the existing Graham's Pond and including most of the Loades North parcel and about half of the Loades South parcel. The northeast half of the Loades South parcel underlain by Cretaceous Great Valley sandstone, shale and conglomerate (map symbol KJgv on Plate 4 - Local Geologic Map) and Jurassic keratophyre and quartz keratophyre tuff formation (map symbol Jsv on Plate 4 - Local Geologic Map). The (inactive) fault bounded southwest side of the valley, including the southwest half of the existing Graham's Pond and Site 3A area is underlain by Franciscan sheared serpentinite matrix mélange (Figure 2).

### 2.2.3 Summary description of Area Bedrock Units[2]

The following are descriptions of bedrock units in the vicinity from available existing documents:

*Wilson Grove Formation* (Twg) is fine grained sandstone with variably abundant silt and some clay matrix; sandstone ranges from clean to saturated by silts and clay. Much of the bedrock has moderate permeability. Some bedrock and much mantle (near surface highly to completely weathered) are expansive. This unit approximately underlies a small portion of the Colella and Briggs property parcels on the northwest end of the overall site area (Plates 4, 13 and 14).

---

[2]Largely from Ellen & Wentworth (1995) and Delattre (2009)

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Franciscan Mélange Unit (map symbol KJfm) is described as sheared/brecciated sandstone and shale including some gouge (highly sheared fault zone material) and may contain hard blocks in varying proportions (hand size to boulder size or larger), is largely not expansive at depth, not expansive to extremely expansive near surface. This unit is shown to underlie the far western portion of the Loades North area, including the southwest half of the existing Graham's pond and the Site 3A area, and about half of Site 3C-2.

*Sandstone, Shale, and Conglomerate* (map symbol KJgv) is predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite. This unit is shown on the Camp Meeker quadrangle geologic map (Plate 4) to approximately underlie a strip of the central portion of the Loades South property area including most if not all of the proposed Site 3C-2, a strip of the central portion of the Loades north property area, and a strip of the southern portion of the Briggs property area.

The Knoxville Formation shale Unit (map symbol Jk) is described as largely mudstone and shale of which both bedrock and mantle are not expansive to significantly expansive. This unit underlies the a small portion of the Loades North parcel but does not appear to underlie Sites 3A or 3C-2.

*Keratophyre Tuff* Unit (map symbol Jsv) is described as deeply weathered breccia's (deposited as extrusive volcanics / pyroclastic materials and then later sheared), most of which is firm but most mantle is expansive. This unit is shown to underlie some of the eastern portion of the Loades South parcel possibly including the eastern extent of the proposed Site 3C-2, the eastern half of the Loades north property area, and the southern portion of the Briggs property area. The bedrock outcrops in the Site 3C-2 area are only slightly weathered and very strong (Figure 3).

*Figure 3: Intermediate keratophyr tuff, dark greenish-gray, slightly weathered, very strong to extremely strong, from surface outcrop west of B-9 by Site 3C-2*

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### 2.2.4 Serpentinite Matrix Mélange

Serpentinite is a rock composed of one or more of a group of complex hydrated serpentinite group minerals. Minerals in this group are formed by serpentinization, a hydration and metamorphic transformation of ultramafic rock, chiefly peridotite, from the Earth's upper mantle. The alteration is particularly prevalent at sea floor tectonic plate boundaries such as the environment that the Coast Range Ophiolite was formed and present in much of the site area as Serpentinite matrix mélange, map symbol spm (Late and Middle Jurassic) and serpentinite rock masses in Franciscan Complex Central Belt Mélange, map symbol KJfm (Late Cretaceous to Early Jurassic).

*Figure 4: View looking west toward more resistant serpentinite surface boulder, likely previously entrained in weak sheared serpentinite matrix which has weathered and eroded away leaving more resistant pod boulder*

The near surface serpentinite underlying much of the site area is generally present as a highly weathered, pervasively sheared, weak to plastic matrix with entrained pods of generally less weathered, closely fractured, weak serpentinite (Figure 2 and Figure 4).

## 2.3 LANDSLIDES

The Camp Meeker Quadrangle map (Delatorre, 2009) Delatorre map, modified and included as Plate 4 depicts a number of Landslides (Historical to Pleistocene), including two large ancient landslides. These mapped landslides and two smaller recent landslide features mapped by M2C adjacent to Graham's Pond (Plate 14) could affect Site 3A. None is located within the construction footprint of Site 3C-2. The landslides noted above are as follows:

- A large ancient mostly evacuated northwest facing landslide extending north from the northern portion of the Loades South parcel, extending down through a portion of the Loades North parcel to the alluvial flatland deposits (Plate 14).
- A large ancient mostly evacuated southeast facing landslide encompassing much of the Colella property and extending south across Graton Road with the toe of the slide approximately marked by Dutch Bill Creek (Plate 14).
- Two smaller recent landslide features on the west side of the existing Graham's Pond and a slump feature to the west of the Site 3A area, (Plate 14).

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# 3.0 SITE 3A SITE INVESTIGATION

## 3.1 SITE 3A SITE DESCRIPTION

The site area can be roughly divided into three geomorphic topographic terranes:

- A gently sloping valley area at about elevation 640 to 670 feet average mean sea level (amsl), trending roughly southwest to northeast along Graton Road and Dutch Bill Creek which encompasses the southern halves of the Colella and Briggs properties on the north side of the valley area, much of the Loades North property and the northern third of the Loades South property (Figure 5). A portion of this area is shown on Plate 14. This area is primarily underlain by alluvium derived from the upland areas to the north and south.

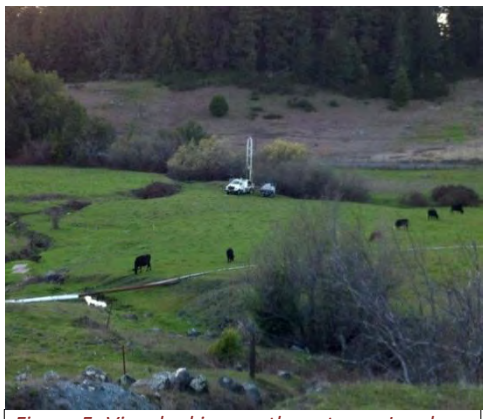

*Figure 5: View looking northwest over Loades North valley area during drilling of boring B-4, with Dutch Bill Creek to the left and Graton Road behind the drill rig and trees*

- A gently rising upland area with approximate 3 : 1 slopes (horizontal to vertical) at about 640 to 750 feet amsl on the north side of the valley, roughly encompassing the northern halves of the Colella and Briggs properties, which is generally underlain by unexposed bedrock units comprising Tertiary sandstone, and Mesozoic Sandstone, shale, and serpentinite matrix mélange.
- A moderately steeply rising upland area with 2 : 1 slopes at about elevation 640 to 890 amsl to the south of the valley area encompassing the southern two thirds of the of the Loades South property, which is generally underlain by near surface and exposed bedrock units of Mesozoic Sandstone, meta-shale, brecciated volcanic tuff and serpentinite matrix mélange. Geologic and topographic maps of the above are presented on Plates 13, 14 & 15.

*Figure 6: View looking north during drilling of boring B-1 at Site 3A*

## 3.2 SITE 3A FIELD INVESTIGATION

### 3.2.1 Site 3A General

A surface and subsurface field investigation program for Site 3A was performed between February 14 and March 7, 2012. The purpose of the field exploration program was to obtain site specific geotechnical information within the proposed development area. M2C performed a site reconnaissance to evaluate general site conditions and possible constraints for planning the overall program and mapped the geologic features and characteristics for the entire project

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area. Our subsurface field exploration program consisted of drilling, logging and sampling four borings (Figure 6) and performing six cone penetration tests (CPTs) within and around the preferred Site 3A area.

A detailed description of our field exploration program is presented below. The surveyed locations of the borings & CPTs are shown on Plate 3, Project Plan and on Plate 7, Cross Sections Plan. The potential new Site 3A reservoir location is shown on Plate 14, Geologic Map of Loades North Property.

A limited initial field investigation for the Site 3C area to explore bedrock characteristics was performed utilizing 6 Test Pits (Figure 7), the details of which are contained in a letter submitted separately and included in Appendix G - Site C Field Investigation Letter.

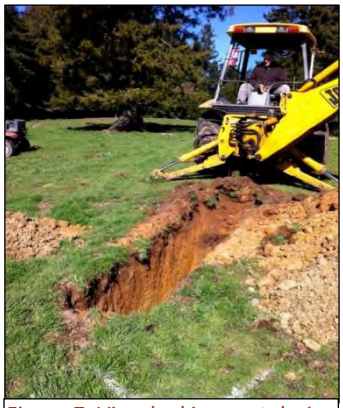

*Figure 7: View looking east during excavation of Test pit TP-4 into soft near surface sandstone in Site 3C-2 area*

### 3.2.2 Site 3A Permitting & Utility Location

Prior to performing the subsurface investigations, M2C visited the site to mark the proposed boring, CPT and Test Pit locations in the field using stakes, flagging and white marking paint. M2C obtained a drilling permit from the Sonoma County Permit and Resource Management Department and notified Underground Service Alert (USA) more than 48 hours prior to our field exploration.

### 3.2.3 Site 3A Geologic Mapping

Geologic mapping of the overall project area primarily focused on identification of slope stability related features such as landslides and slumps, erosional features (Figure 8), and mapping the type and spacial distribution of surface bedrock outcrops. The primary geologic hazards identified during geologic mapping were the presence of two large old landslides previously identified on the published Camp Meeker quadrangle map[3], a modified portion of which is presented on Plate 4, Local Geologic Map,

*Figure 8: View looking east over deeply eroded serpentinite bedrock adjacent to eastside of Graham's Pond*

[3]Delatorre, MP. Koehler, R.D., 2009, Geologic Map of the Camp Meeker 7.5' Quadrangle, Sonoma County, California: California Geological Survey, scale 1:24,000.

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and confirmed in the field by M2C, and the presence of two smaller more recent landslide features and a shallow slump feature located on the west side of the Site A Pond area as shown on Plate 14, Geologic Map of Loades North Property. Geologic Maps of the Briggs & Colella properties and the Loades South property are shown on Plates 13 and 15 respectively.

A generalized summary list of geologic hazards and related construction considerations for the Loades North & South, Briggs and Colella properties is included in Appendix E, Table 1 - Geologic Hazards & Construction Considerations.

A more detailed description of geologic related site characteristics is presented in Section 3, Geology.

### *3.2.4 Site 3A Borings*

The borings were advanced to depths of between 16.5 to 50.25 feet below the existing ground surface (bgs). Samples of the soil and rock materials were retrieved from the borings at selected depths for further evaluation and laboratory testing.

*Figure 9: Fraste Mito 40 track rig during drilling of boring B-4 at Site 3A*

The borings were drilled and sampled using a CME75 drill rig and a Fraste Mito 40 Track rig. The borings were advanced using 6-inch outside diameter (O.D.) augers utilizing the rotary-wash drilling system, which uses constantly circulating drilling mud and facilitates correct blow count data. HQ sized (63.5 mm inside diameter) rotary-wash coring methods were utilized in boring B-3 from 10 feet to the termination depth of 19 feet, but due to the soil-like consistency of the serpentinite bedrock underlying the alluvial soils, Standard Penetration (SPT) and California (Cal) sampling methods were utilized in the serpentinite bedrock materials in borings B-1, B-2 & B-4 to gain blow count information and physical samples of the weak and friable bedrock.

The materials encountered in each boring were visually classified in the field and logged by our field geologist. Soil samples were collected by driving 2 inch outside diameter (O.D.) SPT split spoon samplers and 3.0 inch O.D. Cal split spoon samplers 18 inches into the underlying soil and rock materials using a 140 pound auto hammer falling 30 inches. The number of blows required to drive the sampler was recorded for each 6 inch penetration interval. The number of blows required to drive the sampler the last 12 inches, or the interval achieved where higher resistance was encountered, was recorded as blows per foot and noted on the boring log. Shelby Tube samplers, 18 inch long, 3 inch O.D., were also

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used to gain less disturbed samples of soft clayey soils in borings B-1 & B-2, and hydraulic pressures required to advance the Shelby's were recorded on the logs.

Visual field classifications of the soils encountered in our exploratory borings were made in general accordance with the Unified Soil Classification System (USCS) ASTM D2488 by our staff. The results of our laboratory testing program were used to refine the field classifications. An explanation for the classification of the soils, bedrock and the different sampler sizes is presented on the soil and rock boring legends in Appendix A-1A & A-1B. The logs of the borings for Site 3A are presented on Plates A-2 through A-5 and in Appendix A-2 through A-5.

### *3.2.5 Site 3A Conepenetration Tests*

The CPT program for this investigation consisted of advancing 6 CPT probes (CPT-1 through CPT-6). The CPTs were advanced by Gregg Drilling, Inc. of Martinez, California (Figure 10). The CPT locations are shown on Plate 3, Project Plan and on Plate 7, Geologic Cross Section Plan. All CPTs were advanced to refusal to depths ranging from 7.5 feet to 35 feet bgs.

*Figure 10: CPT Track rig during CPT-4 sounding*

The CPT soundings were performed using a 20-ton capacity cone with a tip area of 15 square centimeters ($cm^2$) and a friction sleeve area of 225 $cm^2$. The cone at the end of the probe measured tip resistance, and a friction sleeve above the cone tip measured frictional resistance. Electrical strain gauges within the cone measured soil parameters continuously for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were transferred to a computer while performing each test. Accumulated data was processed by an onboard computer to provide real-time engineering information, such as the types and approximate strength characterizations of the soils encountered. All CPT soundings were performed in accordance with revised (2002) ASTM standards D 5778-95.

The soil types encountered were determined using the classification chart shown in Appendix C. The CPT plots, showing tip resistance ($Q_c$) and friction ratio ($f_r$%) by depth, as well as interpreted SPT-N, values and detailed methodology descriptions and data are included in are also presented in Appendix C.

Upon the completion of the CPT test holes were grouted using 'tremie' grouting procedures that generally consisted of pushing a hollow CPT rod with a “knock-out” plug to the termination depth of the test

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hole. Grout was then pumped under pressure as the tremie pipe was pulled from the hole. Disruption or further contamination to the site was therefore minimized.

The borings and CPTs were located in the field based on rough measurements or pacing from existing features. The actual locations and elevations were later surveyed in by Sonoma County Water Agency and are reflected on the site plans and boring logs.

## 3.3 SITE 3A LABORATORY TESTING

Laboratory tests were performed on selected soil and bedrock samples to evaluate pertinent geotechnical physical and engineering properties. Soil and rock samples obtained from the subsurface investigation were carefully packaged and sealed and labeled to prevent disturbance and to reduce moisture loss. The soil and rock samples were carefully inspected and reviewed for verification of the field classifications before representative samples from the various strata were selected for laboratory testing. Appropriate tests were selected to assist in subsequent evaluation of material properties for use in analysis. Laboratory tests performed are listed in Table 1A and 1B along with their ASTM designations.

*Table 1A: Laboratory Index Tests and Results Location*

| Test Description | ASTM Designation | Results Location |
|---|---|---|
| In-situ moisture-density | ASTM D2216, D2937 | Plate B-6 & B-12 {App. B} |
| Sieve analysis | ASTM D422 | Plate B-2 & B-5 {App. B} |
| Hydrometer analysis | ASTM D422 | Plate B-2 & B-5 {App. B} |
| Atterberg limits | ASTM D4318 | Plate 14 & Plate B-1 {App. B} |

Index tests were performed on all samples that were tested for shear strength determination. The results of these index tests are also summarized together with the strength test results. The laboratory tests were performed in general accordance with the noted ASTM standards. Consolidation pressures for the Isotropically Consolidated Undrained (ICU) tests were selected based on the estimated overburden pressure at each sample depth. Abbreviated test results for each sample are also included in the boring logs at the appropriate depth.

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*Table 1B: Laboratory Shear Strength Tests and Results Location*

| Test Description | ASTM Designation | Results on |
|---|---|---|
| Unconsolidated Undrained (UU) triaxial compression test | ASTM D2166 | Plates B-6, B-7 & B-8 {Appendix B} |
| Isotropically consolidated undrained (ICU) triaxial compression test with pore pressure measurements | ASTM D2166 | Plates B-9, B-10, B-11, B-12 {Appendix B} |

The interpretation of the laboratory data is presented in Section 5.3 of this report. The results of all of the Laboratory tests are also included on the individual boring logs in Appendix A. Graphic presentations of liquid and plastic limits, particle size distributions, and triaxial shear tests are presented on Plates B-1 through B-12 in Appendix B.

## 3.4 SITE 3A SUBSURFACE CONDITIONS

Based on the information from the borings, CPTs, and laboratory testing, the deepest alluvial deposits at Site 3A area are approximately located near the central axis of the northwest-southeast trending alluvial valley area in the vicinity of borings B-1 & B-4, and CPTs 1, 2, 4 & 6 (Plate 3, Project Plan and Plate 7, Site 3A Cross Sections Plan). In this area, the stratigraphy generally comprises 7 to 14 feet of soft silty clays and clayey sands. Underlying this layer is a loose to dense horizon clayey sand, approximately two foot thick, which is likely continuous between B-2, B-4, CPT-1, CPT-2 & CPT-5 as shown on Plate 8, Cross Section B and Plate 9, Cross Section F. Coarse grained gravels were encountered at the contact between the clayey sand and serpentinite layer in B-2 and the gravely layer was also identified in boring B-2 sample 5A at a depth of 11 feet bgs. Evidence of the presence of this gravely layer is also present in the CPT soundings. The presence of this layer and its extents are uncertain at this time but are not overly significant to the outcome of our analysis.

The alluvial soils described above were underlain by highly to completely weathered, friable to plastic serpentinite bedrock in all borings and CPTs. It should be noted that the soil consistency of the upper serpentinite bedrock strata underlying the alluvial deposits is generally identified as variable very stiff silty and sandy clays and dense clayey and silty sands on the CPT plots in Appendix C. However, the Cone Resistance (qc), Sleeve Friction (fs), Penetration pore pressure (u), Total cone resistance (qt), and Friction Ratio (Rf) data detailed on the CPT plots in Appendix C and most importantly, the visually logged and sampled boring data, is actually generally 'soil consistency' serpentinite bedrock. All CPTs

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were terminated at refusal where the serpentinite became more competent (hard or very dense) with increasing depth. Further bedrock information is presented in Section 3.0. Geology.

Groundwater was encountered at a depths of 7 feet in boring B-1, and 11 feet in boring B-3. This recharged within minutes to about 5 feet in boring B-1, and 9 feet in boring B-3. Per the drilling permit, the borings had to be grouted immediately after drilling, but it is very likely that the actual groundwater table is at about 4-5 feet bgs, which is the approximate level of Dutch Bill Creek, which traverses the site.

Soil and groundwater conditions may differ from those encountered at the exploration locations. If significant variation in subsurface and/or groundwater conditions is encountered during (or after) construction, it may be necessary for M2C to review the recommendations presented. A detailed description of the soil, bedrock and groundwater conditions encountered in our borings and CPTs is presented on the logs of borings and on the CPT plots in Appendix A and C respectively.

### *3.4.1 Summary of Borings and CPT Soundings*

The following Table 2 summarizes the Site 3A field investigation program locations & depths and the best interpretation of the thickness of the alluvial deposits along with the elevation to the top of the serpentinite bedrock layer. The CPTs and borings generally corroborate with one another quite well in terms of establishing the bottom of the alluvial deposits and the top of the serpentinite formation layers. Interpreted strength values from CPT data within the alluvial deposits match the results of the laboratory data in a relative sense. For our analysis, we utilized the material parameters derived from laboratory tests but also relied on CPT data to develop the most appropriate subsurface stratification. CPT soundings were useful in demonstrating the highly variable shear strength of the serpentinite formation.

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*Table 2: Site 3A Boring and CPT Summary*

| Location | Longitude | Latitude | Elevation (FT) | Depth (FT) | Alluvial Deposits thickness or depth to top of Serpentine Bedrock (FT) | Elevation to top of Serpentine Bedrock (FT) |
|---|---|---|---|---|---|---|
| B-1 | W122° 56' 36.02" | N38° 24' 46.75" | 635.56 | 50.0 | 14.0 | 621.6 |
| B-2 | W122° 56' 31.83" | N38° 24' 45.39" | 646.48 | 31.5 | 11.0 | 635.5 |
| B-3 | W122° 56' 31.08" | N38° 24' 49.79" | 642.04 | 19.0 | 7.0 | 635.0 |
| B-4 | W122° 56' 33.99" | N38° 24' 48.36" | 637.81 | 16.5 | 13.0 | 624.8 |
| CPT-1 | W122° 56' 37.00" | N38° 24' 47.39" | 632.25 | 34.8 | 14.0 | 618.3 |
| CPT-2 | W122° 56' 33.71" | N38° 24' 45.75" | 644.19 | 33.8 | 11.5 | 632.7 |
| CPT-3 | W122° 56' 30.80" | N38° 24' 46.03" | 645.01 | 10.3 | 10.0 | 635.0 |
| CPT-4 | W122° 56' 29.74" | N38° 24' 48.53" | 656.12 | 29.4 | 10.5 | 645.6 |
| CPT-5 | W122° 56' 31.07" | N38° 24' 49.56" | 642.86 | 7.5 | 8.0 | 634.9 |
| CPT-6 | W122° 56' 32.33" | N38° 24' 47.21" | 640.82 | 25.4 | 11.5 | 629.3 |

## 3.5 SITE 3A AERIAL PHOTOGRAPHS

Our review of aerial photo stereo-sets from 1980 and 1993 identify remnants of the mapped landslide areas on the Delatorre (2009) map. Both aerial stereo pairs indicate an evacuated and likely re-graded landslide area where the large landslides on the Colella site area referenced above is shown, the toe of which is expressed as the east-west lying portion of Dutch Bill Creek as shown on Geologic Map Plates 13 & 14.

| Film ID | Line | Frame | Scale | Date | Comments |
|---|---|---|---|---|---|
| AV4252 | 210 | 11 & 12 | 1:12,000 | 11/22/93 | Stereo pairs, post dam Const. |
| CIR-SON | 13 | 26 & 27 | 1:24,000 | 04/11/80 | Stereo pairs, post dam Const. |
| M55 AMS | 145 | 4315 & 4316 | 1:24,000 | 09/25/54 | Stereo pairs, pre-dam Const. |
| GS-UX | 1 | 108 & 109 | 1:24,000 | 06/10/52 | Stereo pairs, pre-dam Const. |

Both aerials also show the remnants of the head and west-side side scarp of the mapped landslide extending north from the very northern portion of the Loades South study area toward the Loades North parcel toward Graton Road. The landslide encompasses the northeast facing slope (west side landslide side-scarp remnant of the northwest trending serpentinite outcrop). The hummocky area below the

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Loades residence identified on Plate 15 - Geologic Map of Loades South Property, is a remaining area of complex landsliding and slumping near the headscarp of the old mapped landslide.

The additional smaller landslide features on the west side of the existing Graham's Pond and a slump feature to the west of the Site 3A area noted above were not readily identifiable on the aerial photos. As noted above, these landslide features and other suspected slope instability features identified during field mapping would need to be investigated to confirm/characterize or discount the presence of existing subsurface slide planes.

The aerial photo stereo pairs from 1952 and 1954 are USGS online aerials and are not as clear as the 1980 and 1993 sets but are the only pre-dam construction aerial stereo sets available. They also show the remnants of the mapped landslide areas on the Delatorre (2009) map, and the pre-dam construction topography of the Site 3A and existing Graham's Pond valley area prior to the diversion of Dutch Bill Creek. The original Creek location appears less meandering and ran approximately northwest-southeast at the approximate center of the existing dam abutment. The upland areas in the Loades North & South parcels appear more vegetated in the pre-dam aerials.

For general reference, copies of one of each of the four stereo sets are included in Appendix F, Aerial Photographs.

## 3.6 SITE 3A GEOLOGIC SECTIONS

Two geologic sections in the Site 3A pond area derived from subsurface data from borings B-1, B-2 and CPTs 1, 2, 4 & 6, are presented on Plates 8 & 9 and the Section's Plan on Plate 7. These sections show M2C's interpretation of the subsurface stratigraphy in the Site 3A pond area and were used in our analysis.

## 3.7 SITE 3A SEISMIC CONDITIONS

The site is not underlain by known active faults (i.e., faults that exhibit evidence of ground displacement in the Holocene period or last 11,000 years), nor does the project lie within an Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, 1997 and CDMG, 1982). Therefore, the potential for ground rupture due to faulting through the structure is considered low. The San Andreas fault is located approximately 12.16 km (7.6 miles) to the west of the site and is the design fault for the site area the project site (Moment Magnitude, $M_w$7.9), and thus design will conform to Active Fault Near-Source Zone requirements in the 2007 California Building Code (CBC).

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# 4.0 SITE 3C-2 SITE INVESTIGATION

## 4.1 SITE 3C-2 SITE DESCRIPTION

*Figure 11: View looking approximately north over rock outcrops on the gently sloping deforested central Site 3C-2 area toward the forested and gently to steeply sloping northern extent of the site*

The proposed Site 3C-2 new reservoir grading area is located toward the southeast end of the Loades South property parcel (Plate 2), and occupies the highest terrain in the overall four property parcel project area at 890 average mean sea level (amsl). The parcel is a moderately steeply rising upland area ranging from about elevation 640 amsl in the north of the parcel to 890 amsl in the southeast portion of the parcel where the proposed new Site 3C-2 reservoir is situated. The greater site area is generally underlain by one to two feet of poorly developed topsoil and colluvium over near surface bedrock units with small and scattered low lying bedrock outcrops of Mesozoic sandstone, volcanic tuff, serpentinite matrix mélange and high grade metamorphic rocks and breccia's (Figure 11). The approximate southern half of the Site 3C-2 area slopes gently to the south, moderately to the west, is deforested with poorly developed topsoil and currently used to graze free roaming cattle. The northern half of the Site 3C-2 area is forested with redwood, bay, fir and pine trees and slopes gently to the north and west and then steeply in the far north and west with slopes up to 2 : 1. Investigation methods for Site 3C-2 are described in this section and the analyses for Site 3C-2 are presented in section 6.

*Figure 12: View looking at dark grey tuff intrusion at center of TP-7 with yellow-green serpentinite at lower left and baked zone meta-sediments at lower right of tuff*

## 4.2 SITE 3C-2 FIELD INVESTIGATION

### 4.2.1 Site 3C-2 Permitting & Utility Location

Prior to performing the subsurface investigations for Site 3C-2, the proposed boring, test pit and seismic refraction line locations were marked in the field using stakes, flagging and white marking paint. A drilling permit from the Sonoma County Permit and Resource Management Department (Sonoma County PRMD) was obtained and Underground Service Alert (USA) notified more than 48 hours prior to commencing subsurface field exploration activities.

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### 4.2.2 Site 3C-2 Geologic Mapping

Available published geologic and geotechnical reports and maps were reviewed. including summary subsurface test pit data from the preliminary investigation.

An initial field reconnaissance was performed and then the geology of Site 3C-2 and surrounding area was mapped utilizing a 1-foot contour newly surveyed topographic base map provided by BRCE.

*Figure 13: One of few reliable bedding measurements taken in a layer of interbedded sandstone conglomerate in TP-16*

#### 4.2.2.1 Bedrock Outcrop Mapping

Geologic mapping focused on identification of and mapping the type and spatial distribution of numerous small surface bedrock outcrops with emphasis on ensuring that serpentinite does not occur within the planned excavation.

#### 4.2.2.2 Structural Field Data

Attempts were made to record structural measurement data from the various lithologies in the site area (Figure 13). However definite and repeatable structural measurements such as bedding dips and strikes, joint and fracture set orientations, consistent foliations and numerous other linear features were not present, definite and repeatable in the site area.

#### 4.2.2.3 Slope Stability

Possible slope instability geomorphic features, especially on the steeper colluvial slopes flanking the northern end of the site, were inspected closely for features that may have indicated ancient, historic or recent movement potential such as landslides, slumps, natural or accelerated creep or erosional features. None were observed. It should be noted that the large, ancient landslide shown on Plate 15 and discussed in section 2.3, does not impinge on the Site 3C-2 area as the eroding headscarp is approximately 380 feet NNW of the northern extent of the Site 3C-2 grading area.

*Figure 14:: Typical highly weathered, weak to friable shale bedrock from boring B-9, cored from 0 to 19 feet. RQD <25%*

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A generalized summary list of geologic hazards and related construction considerations which includes the Loades South property is included in Appendix E – Table 1 - Site 3A General Geohazards & Construction Considerations. A more detailed description of geologic related site characteristics is presented in the Section 3, Geology, of this report.

### *4.2.3 Site 3C-2 Test Pits*

#### *4.2.3.1 Site 3C-2 Preliminary Test Pits TP-1 through TP-6*

Subsurface data from the existing limited subsurface investigation that utilized test pit excavation methods was used to constrain the location of the various rock type boundaries, identifying the depth to bedrock and the probable rippability of the bedrock. The test pits are shown on Plate 3, Site Plan Topographic Map (TP-1 through 6), and the potential new reservoir location is shown on Plate 15.

#### *4.2.3.2 Site 3C-2 Test Pits TP-7 through TP-18*

A total of 18 test pits were excavated in the southern portion of the Loades South parcel to better define the Site 3C-2 geologic formation boundaries, and particularly the serpentinite formation boundary and the sandstone / conglomerate (map symbol KJgv), (Plate 15).

### *4.2.4 Site 3C-2 Borings*

The borings were advanced to depths of between 31.5 to 60 feet below the existing ground surface (bgs). HQ Size (2½ inch core diameter; 3 25/32 inch hole diameter) rotary-wash rock coring and California Sampler (2½ inch inside diameter; 3 inch outside diameter) interval soil sampling methods were utilized. Samples of the soil and rock materials were retrieved from the borings at selected depths for further evaluation and laboratory testing.

Borings B-5 through B-11 were drilled and sampled using a truck mounted Failing B53 Mud Rotary drill rig. Borings B-12 & B-13 were drilled and sampled using a limited access 'Marl M5 Track Rig' ('Rhino rig'), because Boring B-12 was located in a steeper forested area. The borings were advanced using a tri-cone drill bit and 10 foot long, hollow drill string sections, utilizing the rotary-wash drilling system which uses constantly circulating drilling mud and facilitates more accurate blow count data due to having minimal pressure differential inside and outside the hollow drill string rods. The drilling method was switched from modified California interval soil sampling methods to HQ sized continuous rock coring methods depending on apparent bedrock rock quality.

The materials encountered in each boring were visually classified and logged by our geologist in the field during drilling. Soil samples were retrieved by driving a 3.0 inch O.D. California split spoon sampler containing three cylindrical six inch long brass soil samplers 18 inches into the underlying soil and rock

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materials using a 140 pound auto hammer falling 30 inches. The number of blows (n-counts) required to drive the 18 inch long California sampler the three 6 inch intervals was recorded. The number of blows required to drive the sampler the last 12 inches, or the interval achieved where higher resistance was encountered, was recorded as blows per foot and noted on the boring log.

Field classifications of the soils encountered in our exploratory borings were made in general accordance with the Unified Soil Classification System (USCS) ASTM D2488. The results of the laboratory testing program and review of samples were used to refine the field classifications. The soil boring and rock coring log legends Plates A-1A & A-1B provide fundamental information for the classification of soils and bedrock using the USCS. Site 3C-2 boring logs B-5 through B-13 are presented on Plates A-6 through A-14 in Appendix A (Site 3A boring logs B-1 through B-4 are presented on Plates A-2 through A-5 in Appendix A).

### *4.2.5 Site 3C-2 Seismic Refraction Lines*

*NorCal Geophysical Consultants, Inc. of Cotati, Sonoma County provided seismic refraction survey of the site under subcontract. Their summary report survey for Site 3C-2 is included as Appendix D.*

The primary purpose of including a seismic refraction survey in the subsurface exploration was to evaluate the possibility of the occurrence of hard blocks of rock in the excavation area. This was a potential concern because large tuff inliers and exotic blocks are present as surface outcrops to the east and west of the Site 3C-2 grading area. These outcrops are slightly weathered to almost fresh and very strong to extremely strong and could affect excavatability of the site.

## *4.3 SITE 3C-2 LABORATORY TESTING*

Laboratory tests were performed on selected soil and bedrock samples to evaluate pertinent geotechnical physical and engineering properties. Soil and rock samples obtained from the subsurface investigation were carefully packaged, sealed and labeled to prevent disturbance and to reduce moisture loss. The soil and rock samples were inspected to verify or update initial field classifications prior to representative samples from the various strata being selected for laboratory testing. Appropriate laboratory tests were selected to assist in subsequent evaluation of material properties for use in engineering analysis. Laboratory tests performed are listed on Tables 3A and 3B along with their ASTM designations.

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*Table 3A: Laboratory Index Tests and Results Location*

| Test Description | ASTM Designation | Results Location |
|---|---|---|
| In-situ moisture-density | ASTM D2216, D2937 | Boring logs; Plates B-13 through B-19 {Appendix B} |
| Sieve analysis | ASTM D422 | Boring logs; Plate B-23 {Appendix B} |
| Atterberg limits | ASTM D4318 | Boring logs; Plate B-22 & B-22 {Appendix B} |

Index tests were performed on all samples that were tested for shear strength determination. The results of these index tests are summarized together with the strength test results. The laboratory tests were performed in general accordance with the noted ASTM standards. Abbreviated test results for each sample are also included in the boring logs at the appropriate depth. The interpretation of the laboratory data is presented in Section 6.3 of this report.

*Table 3B: Laboratory Shear Strength Tests and Results Location*

| Test Description | ASTM Designation | Results Location |
|---|---|---|
| Triaxial Shear Test (confined-undrained with pore pressures) | ASTM D4767 | Boring logs; Plates B-19, B-20 {Appendix B} |
| Unconsolidated Undrained (UU) triaxial compression test | ASTM D2166 | Boring logs; Plates B-13, B-14, B-15, B-16, B-17, B-18 {Appendix B} |

The results of all of the Laboratory index tests are also included on the individual boring logs in Appendix A at the sample depth. Graphic presentations of liquid and plastic limits, particle size distributions, and shear tests are presented in Appendix B.

## 4.4 SITE 3C-2 SURFACE & SUBSURFACE CONDITIONS

The near surface (upper foot) soils in the proposed construction area typically consisted of organic rich colluvium (top soil) underlain primarily by sandstone conglomerate bedrock which was typically highly weathered, friable, with a low to moderate plastic consistency and had to be drilled and sampled using the 'California sampler' soil sampling method. The materials encountered had "blow counts" that were typically 30 to 40 blows per foot (medium-dense to dense). The sandstone conglomerate varied in thickness from 24 to 41 feet within in the proposed Pond 3C-2 footprint and, although comprising primarily re-lithified sandstones, including clasts and horizons of volcanic tuff, shale and metamorphic

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rocks. Underlying the sandstone conglomerate layer were generally light to dark gray sedimentary rocks consisting of poorly sorted graywacke sandstones with intervening horizons of shale and meta-shale. The competency of these graywacke sequences was highly variable from friable to plastic to moderately strong to strong. The rare strong rock horizons were typically meta-shales and some meta-graywacke's.

Outcrops of dark gray, slightly weathered to fresh, strong to very strong volcanic tuff which appear to be present as exotic inclusions in the general site area, were encountered northeast of a serpentinite/sandstone conglomerate formational boundary that is oriented approximately northwest/southeast, a few feet southwest of the proposed pond 3C-2 footprint. The outcrops vary considerably from large boulder size to house size; however, there are appear to be no tuff surface outcrops actually within the proposed pond 3C-2 footprint. A house sized tuff block surface outcrop is present about 200-300 feet to the east of the east side of the proposed pond 3C-2 footprint and a 20-foot or so diameter surface outcrop is located a few feet to the southwest of the southwest side of the pond 3C-2 footprint.

Serpentinite bedrock was present just a few feet to the southwest of the proposed pond 3C-2 construction footprint and was encountered to the depths explored in borings B-11 and B-13, which were outside the footprint. Additional test pits were excavated to help better define the serpentine/sandstone conglomerate formational contact boundary. The serpentinite/sandstone conglomerate contact was unearthed in test pit TP-7 and there appears to be little transitional rock assemblage.

A detailed account of subsurface conditions encountered in our Site 3C-2 borings is presented in Appendix A, Site 3C-2 Boring Legend & Boring Logs B-5 through B-13. A detailed account of the near-surface (4 to 10 feet) conditions encountered in our Site 3C-2 and vicinity test pits is presented in Appendix H – Site 3C & 3C-2 Logs of Test Pits TP-1 through TP-18.

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## 4.5 SITE 3C-2 AERIAL PHOTOGRAPHS

Our review of the 1952, 1954, 1980, & 1993 aerial photo stereo-sets as detailed above with respect to the Site 3C-2 grading area and immediate vicinity found that that there appears to have been no apparent land use change during that 41 year period or to the present.

| Film ID | Line | Frame | Scale | Date | Comments |
|---|---|---|---|---|---|
| AV4252 | 210 | 11 & 12 | 1:12,000 | 11/22/93 | Stereo pairs, post Loades dam Const. |
| CIR-SON | 13 | 26 & 27 | 1:24,000 | 04/11/80 | Stereo pairs, post Loades dam Const. |
| M55 AMS | 145 | 4315 & 4316 | 1:24,000 | 09/25/54 | Stereo pairs, pre-Loades dam Const. |
| GS-UX | 1 | 108 & 109 | 1:24,000 | 06/10/52 | Stereo pairs, pre-Loades dam Const. |

We researched and commented on the large mapped mostly evacuated landslide remnant in the central portion of the Loades South Property and on other slope stability related features in the four parcels. Please refer to section 3.5 Site 3A Aerial in this report for the above. The only additional comment is that the closest part of the receding (eroding) landslide headscarp, as shown on Plate 15 Geologic Map of Loades South Property, is about 250 linear feet from the north edge of the proposed construction area for Site 3C-2.

## 4.6 SITE 3C-2 GEOLOGIC SECTIONS

The two geologic cross sections G-G' and H-H' , which approximately traverse the long and short axis of the proposed reservoir at right angles are derived from subsurface boring and test pit data (Plate 11). These sections show interpretation of the subsurface stratigraphy in the Site 3C-A pond area and were used in the geotechnical and seismic analyses.

## 4.7 SITE 3C-2 SEISMIC CONDITIONS

The site is not underlain by any known active faults (i.e., faults that exhibit evidence of ground displacement in the Holocene period or last 11,000 years), nor does the project lie within an Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, 1997 and CDMG, 1982). Therefore, the potential for ground rupture due to faulting through the structure is considered low. The San Andreas fault is located approximately 12.16 km (7.6 miles) to the west of the site and is the design fault for the site area the project site (Moment Magnitude, $M_w$7.9), and thus design will conform to Active Fault Near-Source Zone requirements in the 2007 California Building Code (CBC).

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# 5.0 SITE 3A ANALYSIS, CONCLUSIONS & RECOMMENDATIONS

## 5.1 SITE 3A GROUND MOTION STUDY

The purpose of this study was to develop the design response spectrum, for the Pond Site 3C-2 located in near Occidental, California

Site Location: Latitude: 38.412811 Longitude: -122.942677

The most significant seismic event to affect the project site would likely be a maximum moment magnitude Mw 7.2 earthquake resulting from the San Andreas-Rodgers Creek fault zone, or a moment magnitude Mw 7.9 earthquake in the SAF. The Maximum Considered Earthquake (MCE, or 2% probability of exceedence in 50 years) is considered to rupture the entire San Andreas system with a magnitude of M 7.9.

The proposed Site 3A embankments rest on an average of 10 feet (for site response purposes) of alluvial clayey deposits overlying serpentinite bedrock. For the planned analysis of the embankment, ground motions were computed for a soft rock outcrop site. Based on an analysis of values at rock sites which have recorded strong ground motions, a typical shear wave velocity (*S)* (30m) value for soft rock is about 550m/sec (Silva et al 1999, See Plate 15).

M2C developed a horizontal component deterministic design response spectrum (5% damped) at the 50$^{th}$ percentile at the Proposed Site 3A embankments. Alternatively, an 84$^{th}$ percentile spectrum can be used if this were deemed a “high hazard dam” per DSOD guidelines.

The next Generation Attenuation relations (NGA, 2008) developed by Abrahamson and Silva, Boore and Atkinson, Campbell and Bozorgnia, and Chiou and Youngs were used to compute estimates of median sigma ground motions. These four relations were selected as they have characterized site conditions in terms of $S_v$ (30m) in a consistent manner. Additionally, several of the relations have included depth to 1 km/sec (or 2.5 km/sec) material as independent variables to more accurately characterize strong ground motions at sites with shallow depths to bedrock materials, as occurs at the Site 3A embankments. The NGA relations (NGA, 2008) provide median estimates of 5% damped pseudo absolute response spectra over the period range of 0.01 sec to 10.00 sec. Plate 16 and 17 show the spectral ordinates and target response spectrum plots respectively. The estimated peak ground acceleration at the site from the MCE on the San Andreas Fault is 0.517g.

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## 5.2 SITE 3A LIQUEFACTION POTENTIAL

Liquefaction is a secondary effect of earthquake shaking that is caused by cyclic stress that increases pore-water pressure in granular soils. As a result of the increased pore-fluid pressure, the soil undergoes a substantial loss of strength and can deform and, if the soil mass is not confined, acquire mobility sufficient to permit both horizontal and vertical displacement. Soils most susceptible to liquefaction are generally saturated, loose, clean (low clay content), uniformly graded, younger than 15,000 years, fine-grained sand or silt deposits.

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral displacement of subsurface liquefiable material or sensitive clays. This phenomenon typically occurs adjacent to free faces such as creek channels, harbors and canals. Because the site is considered to have low potential for liquefaction, potential for lateral spreading is also considered to be low. However, the possibility of shaking-related random ground cracking or seismically induced slope failure affecting the site and surrounding area should not be precluded.

Plate B-2 in Appendix B shows the results of the Atterberg limits tests. Plate 14 shows all the results of the laboratory tests and indicates that the materials tested generally can be classified as CL (lean clay) and CH (fat clay). The plasticity chart has three zones: A, B, and C, according to Seed (2003). These zones are shown in the figure below and defined as follows: The 10 Atterberg Limits performed on the materials indicated the average Liquid Limit was over 45% and plastic limit well above 20% as shown on Plate 14. The materials were generally CL and some CH and are well outside the “B Zone”. The field moisture contents for all 10 samples tested were above 85% of the Liquid Limits however since none of the samples fell under the B-Zone Criterion the evaluation of 0.85* water content is not applicable.

Recommendations Regarding Assessment of “Liquefiable” Soil Types (Seed et. al. 2003)

- Zone A: Considered potentially susceptible to classic cyclically induced liquefaction.
- Zone B: In some cases susceptible to liquefaction (if the in-situ water content is greater than 85% of their liquid limit).
- Zone C: Generally not susceptible to cyclically induced liquefaction (above Zone B in Figure shown).

Based on the above evaluations and limited samples collected, M2C **does not** anticipate that the clayey alluvial deposits (ranging in thickness from 7 to 14 feet) underlying Site 3A will undergo liquefaction or significant strength loss during earthquake loading.

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## 5.3 SITE 3A RESERVOIR MATERIAL PROPERTIES

### 5.3.1 Site 3A General

The proposed reservoir and its foundation soils consist primarily of three types of materials:

1. Reservoir Fill (Proposed)
2. Native Alluvial Deposits
   (1) Soft Silty Clays
   (2) Clayey Sands
3. Serpentinite Bedrock

Plates 8 and 18 show the site plan, profile, and cross section of the dam along with the subsurface stratigraphy. The reservoir fill materials are assumed to be compacted clays obtained from the onsite alluvial deposits which are lean to fat clays. It should be noted that these high plasticity clays will need to be placed with great care as far as the placement moisture and degree of compaction to achieve optimal long term performance is concerned. The serpentinite bedrock has the consistency of interbedded medium stiff to very stiff silty clays to medium dense to dense clayey sand.

#### 5.3.1.1 Classification and Index Tests

Grain size distribution and Atterberg limits determinations were performed on numerous samples of the alluvial material, and a few tests were performed on the serpentinite bedrock. Index properties of the materials are summarized in Table 4 below:

*Table 4: Average of the Index Test Results*

| Material Description | Moisture Content % | Total Density (PCF) | Dry Density (PCF) | Void Ratio | Saturation (%) | Specific Gravity |
|---|---|---|---|---|---|---|
| Dam Fill Proposed[1] | 15 | 126.5 | 110 | NA | NA | 2.70 |
| Soft Silty Clays[2] | 30 | 120 | 92.3 | 0.9 | 0.98 | 2.70 |
| Clayey Sands[3] | 29.5 | 118.8 | 91.7 | 0.8 | 0.95 | 2.70 |
| Serpentinite Bedrock[4] | 18.0 | 140 | 121.7 | 0.4 | 0.8 | 2.65 |

1. Assumed material parameters
2. Average of B-1, Sample 1A, B-2 Sample 3A, and B-3 Sample 1C

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3. From B-3 Sample 5C
4. See Hoek-Brown Failure criterion development Table 6

Based upon review of the laboratory testing results, the native materials are expected to exhibit shear strength properties lower than the dam fill materials.

### 5.3.2 Shear Strength

To estimate the shear strength of native alluvial deposits and serpentinite formation bedrock materials, two types of laboratory testing were performed:

1. Isotropically Consolidated Undrained (ICU) triaxial compression tests with pore pressure measurements.
2. Unconsolidated Undrained (UU) triaxial compression tests.

In general, the results of the ICU tests provided the most appropriate results and were most suitable for use in the analysis, but the UU tests were also considered in arriving at our results. Results from the ICU tests were used in developing shear strength parameters for the fill, native materials, and bedrock.

#### 5.3.2.1 Effective Shear Strength Failure Envelope

The static shear strengths of the dam fill and foundation materials were estimated using ICU triaxial compression tests with pore water pressure measurements. Plates B-9, B-10, B-11, B-12 in Appendix B show the effective strength failure envelope. The effective material strength parameters used in the analysis are shown on Table 5A.

***Table 5A: Effective Shear Strength Parameters***

| ***Material*** | ***Moist Unit Weight (pcf)*** | ***Cohesion Intercept 'c' (psf)*** | ***Friction Angle*** |
|---|---|---|---|
| **Dam Fill (Proposed)** | 126.5 | 200 | 30 |
| **Soft Silty Clays** | 120 | 0 | 23.3 |
| **Clayey Sands** | 118.8 | 0 | 31.1 |
| **Serpentinite Bedrock** | 140 | Hoek-Brown | |

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#### 5.3.2.2 Total Shear Strength Failure Envelope

For the seismic (pseudo-dynamic) analysis, undrained shear strength was used. For this study, we used the static total strength parameters from the ICU tests. Plates B-9, B-10, B-11, B-12 in Appendix B show the total strength failure envelope for the materials, and weathered bedrock, respectively.

Clayey soils exhibit an increase in shear strength during rapid or seismic loading cases. The phenomenon of increase in strength for clayey materials has been well-documented by previous studies and is a generally accepted standard of practice (Duncan and Wright, 2005). Typically, a shear strength increase of 20 percent or greater can be used for earthquake loading conditions for clayey soils. In this report, M2C took a more conservative approach, and did not apply any shear strength increase due to the size of the embankment and the anticipated embankment deformations. Stability analyses were performed using no strength increase and are presented in Section 5.4.

***Table 5B: Total Shear Strength Parameters***

| ***Material*** | ***Moist Unit Weight $\gamma_m$ (pcf)*** | ***Cohesion Intercept c (psf)*** | ***Friction Angle $\phi$ (degrees)*** |
|---|---|---|---|
| Dam Fill Proposed | 126.5 | 300 | 28.0 |
| Soft Silty Clays | 120.0 | 300 | 18.0 |
| Clayey Sands | 118.8 | 200 | 20.0 |
| Serpentinite Bedrock | 126.5 | Hoek-Brown | |

### 5.3.3 Hoek-Brown Criterion Input Parameters

The rock mass shear strength parameters resulting from a Hoek-Brown evaluation are generated using a specific method based on research by Dr. Hoek and Dr. Brown. The Hoek-Brown Strength Criterion is an empirical rock strength criterion which takes into consideration intact rock strength, as well as influence of the discontinuities within the rock mass on the strength of the mass. Because rock mass shear strength is difficult to measure in the laboratory, (i.e. large samples are difficult to sample and test and will not contain a sufficient number of discontinuities to represent the rock mass), the rock mechanics community has adopted this approach in slope stability analysis where kinematics of the rock do not dictate the rock mass stability. The generalized Hoek-Brown strength parameters were developed for the serpentinite formation bedrock materials using our best estimates for intact rock strengths, 'Rock

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Mass Rating' (RMR), 'Geological Strength Index' (GSI), material composition index and rock disturbance estimates. It should be noted the primary emphasis and scope of this investigation was focused on obtaining shear strength parameters of the alluvial deposits. Future studies that include utilizing the serpentinite materials as direct foundation materials will require more detailed characterization of the serpentinite shear strength parameters.

*Table 6: Generalized Hoek-Brown Strength Parameters for Serpentinite Bedrock*

| Material | Total Unit Weight, (pcf) | GSI | Mi | Disturbance Factor | Rock Strength (psf) | Mb | s | a |
|---|---|---|---|---|---|---|---|---|
| Serpentinite Bedrock | 140.0 | 5 | 0.0 | 0.5 | 12000 | 0.29 | 0.9 | 0.53 |

## 5.4 SITE 3A STABILITY EVALUATIONS

### 5.4.1 General Approach to Site 3A Evaluation

Conventional limit equilibrium slope stability and earthquake-induced deformation methods were used to assess the performance of the dam. Static and pseudo-dynamic stability evaluations were carried out for both the upstream and downstream slopes of the dam. Effective shear strength parameters were used in the static stability analysis. Undrained shear strength parameters, developed as described in Section 6.0, were used in the pseudo-dynamic stability analysis.

Slope stability analyses were performed using the computer program Slide 5.0 (www.rockscience.com, 2008). The program uses two-dimensional limit equilibrium technique and the method of slices, and allows the user to select various methods including simplified Bishop, simplified Janbu, Spencer, Morgenstern-Price, etc., to calculate the minimum factors of safety for both circular and block failure surfaces. The results presented for the upstream and downstream cases represent the surface type yielding the least factor of safety. Slope stability analyses presented in this report were performed using the Spencer Method. In the Spencer Method, compatibility of both moment and force equilibrium is enforced, which provides a higher level of reliability than methods that compute moment or force equilibrium separately.

### 5.4.2 Reservoir Profile and Cross-Section

A representative reservoir profile (Section B-B') and cross-section was chosen to best represent the surface and subsurface conditions over the entire site. The cross-section analyzed and shown on Plate 17 is considered to represent the most critical area at the maximum height of the embankment. Plates

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18 to 23 show the cross-sections and stability analysis results from the study. A water storage elevation of 650 feet amsl was used in our analysis which is 3.5 feet below the proposed embankment crest elevation. We have assumed that the embankment performance should allow for a minimum of 1 foot of free-board after the stressing events (static consolidation and earthquake induced deformations).

### 5.4.3 Long-Term Static Stability

For long-term static stability, a drained analysis with assumed steady state seepage conditions was performed for both the upstream and downstream slopes of the dam. For each slope condition, both deep and shallow failure conditions were evaluated. The analyses were performed using unit weights and effective strength parameters of the materials as shown in Table 5A and 5B.

The results of the long-term static stability analysis are shown on Plate 21 for the downstream slope and Plate 23 for the upstream slope. The lowest factor of safety against long-term static stability for the upstream and downstream slopes was 1.3 and 2.1, respectively.

Plate 22 shows the use of vertical reinforcing elements in the analysis that provide shear reinforcement to the slope for the long term drained conditions. As shown on Plate 21 the long term slope does not meet the required permanent factor of safety of 1.5. The vertical reinforcing elements are modeled to be geopier (www.geopier.com) elements that could be 1.5 foot diameter rammed aggregate piers embedded about 10 feet into the serpentinite bedrock and placed at a 5 X 5 foot grid spacing. The additional shear strength gained from reinforcement of the potential slide plane at this grid spacing can conceptually provide the required factor of safety of 1.5. It should be noted this analysis was performed only for conceptual purposes and will require detailed additional study if it is deemed appropriate in the future. This analysis does not address the long term consolidation settlement of the alluvial deposits that will occur as a result of embankment loading at the proposed site.

The rammed aggregate piers are one possible ground improvement solution for Site 3A although may not prove practical due to development cost issues. Other ground improvement solutions could include deep soil mixing with additives to improve soil strength. A third option that could provide satisfactory dike stability would be to remove the weak and expansive clays underlying the dikes and prepare a pond configuration with reduced overall dike heights. Brelje & Race evaluated the third option in the 30 % Conceptual Design Report to allow cost comparison with development at other pond sites.

### 5.4.4 Short-Term Seismic Stability

A pseudo-dynamic approach was utilized for evaluation of slope stability under seismic conditions. In this approach, earthquake forces are represented by an equivalent dynamic horizontal force which consists of the estimated effective horizontal ground acceleration multiplied by the mass of the potential

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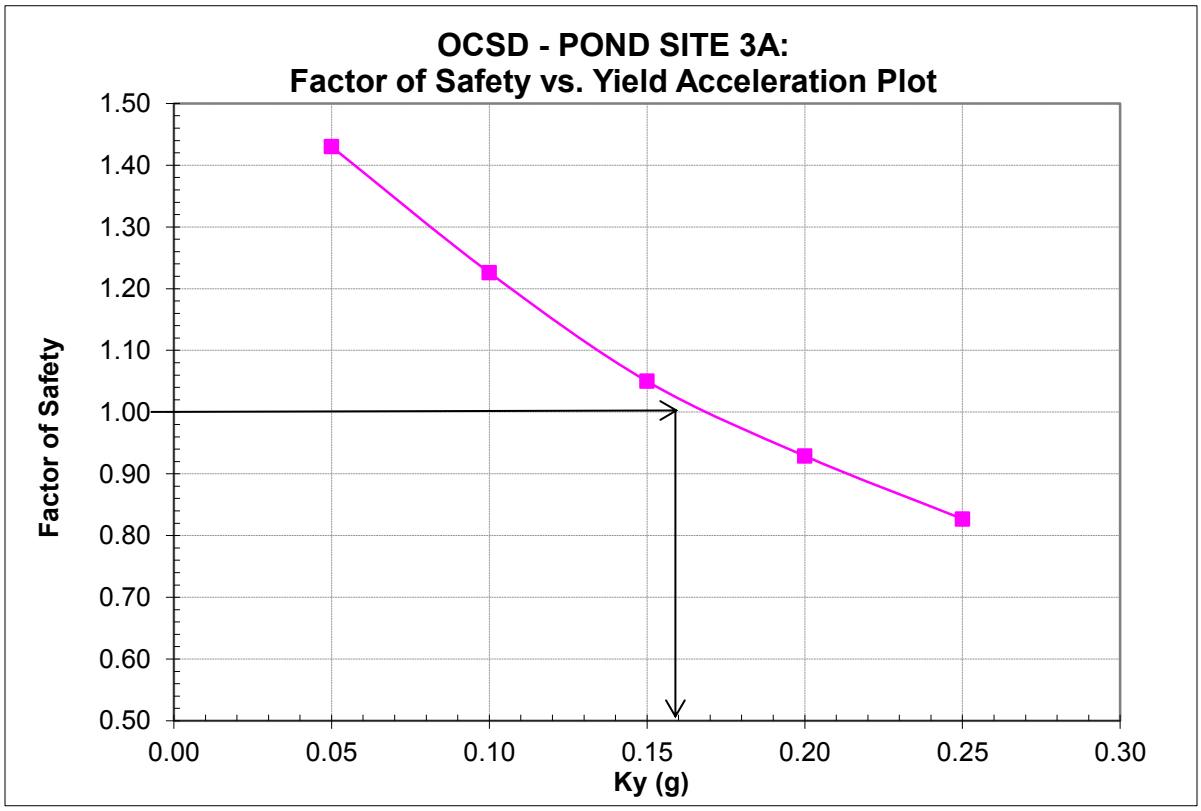
slide material. For earthquake loading conditions, the analyses were performed using static total strength parameters as the undrained shear strength for the materials, as shown in Table 5B and Plates B-9, B-10, B-11, B-12.

Based on the in-situ testing, laboratory index properties and shear strength test results, the alluvial clay materials and serpentinite bedrock are not anticipated to undergo strength loss during seismic loading. Typical strength increases for rapid loading can be 20 percent or greater; however, a conservative approach of no strength increase was used for this project.

In the pseudo-dynamic stability analysis, the result is the yield acceleration, $K_y$, an effective horizontal ground acceleration at which a potential sliding surface would develop a factor of safety equal to unity. The results of the pseudo-dynamic stability analysis are shown on Plate 24 for the downstream slope. The yield acceleration under seismic conditions is 0.17g for the downstream slopes. The figure below provides a summary of the downstream factor of safety and its variation with the applied horizontal seismic coefficient.




**Factor of Safety vs. Yield Acceleration**

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## 5.5 SITE 3A DEFORMATIONS

Earthquake-induced deformations of the embankments were estimated using the simplified procedures by Bray and Travasarou (2007); which basically use the concept of a rigid block sliding along a failure plane within an embankment slope that was originally suggested in the Newmark's Method. In the sliding block approach, a failure slope wedge is idealized as a rigid mass above a plane of sliding with the earthquake induced accelerations. If the acceleration levels for the sliding block are lower than the yield acceleration for the sliding block, no movement will occur along the sliding plane. On the other hand, if the acceleration levels are higher than the yield acceleration, there will be net movements between the sliding block and the base, i.e., displacements. The Bray and Travasarou (2007) approach is a simplified semi-empirical method to estimate deviatoric-induced slope displacements based on the results of nonlinear fully coupled stick-slip sliding block analyses using a comprehensive database of hundreds of recorded ground motions. Plate 26 shows the results of the analysis. Given the Ky=0.17, there is only a 0.0003% chance of zero to one centimeter of displacement and we estimate the horizontal earthquake induced deformations to be 67 cm or about 2.2 feet. The resulting crest settlement can be estimated to be about 70% or about 1.54 feet.

*Table 7: Input Parameters for Site 3A Deformation Analysis*

| Input Parameters | Value |
|---|---|
| Yield Coefficient (ky) | 0.17 g |
| Initial Fundamental Period (Ts) | 0.10 seconds |
| Degraded Period (1.5Ts) | 0.15 seconds |
| Moment Magnitude (Mw) | 7.9 |
| Spectral Acceleration ( Sa(1.5Ts) | 1.142 g |

## 5.6 SITE 3A SHORT-TERM RAPID DRAWDOWN STABILITY

Total shear strength parameters were used in evaluation of the rapid drawdown stability using Slide. The analysis was carried out by changing the position of the phreatic surface in the cross-section to model the change or removal of the reservoir water level. The total strength parameters were used for the rapid draw-down analysis. The results of the rapid draw-down stability analysis for the upstream slopes with different reservoir water levels are shown on Plate 25. The minimum temporary factors of safety are 2.5 and 3.1 for the northwest and southeast embankments respectively and these are well

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above a factor of safety of 1.2 which is typically the minimum required factor of safety (USACOE, Slope Stability Manual, 2003).

## 5.7 SITE 3A CONCLUSIONS AND SUMMARY OF DAM PERFORMANCE RECOMMENDATIONS

The dam fill materials will likely be constructed using high plasticity clays obtained from the on-site alluvial clays. The underlying clayey alluvial deposits and serpentinite formation bedrock materials are not likely to liquefy and therefore will not undergo significant strength loss due to a seismic event.

No known active fault traces are present at the site, thus earthquake-induced ground rupture is not anticipated. Landslide maps and M2C field mapping show areas of instability at the site and surrounding areas. The required minimum factor of safety for long-term steady state conditions is 1.3 for the downstream side of the proposed northern dike and mitigation will be required to increase this to the required 1.5.

The results of the seismic stability analysis show that the proposed embankments will likely be subjected to ground shaking from the maximum credible earthquake of magnitude 7.9 on the San Andreas Fault. The deformation analysis indicates about 2.2 feet of slope deformation and 1.54 feet of crest settlement. The pond is assumed to be operated at a 650 feet amsl water level for this study and we assume that a 3.5 feet of operational free-board can be maintained. Uncontrolled release of water is not expected from a major seismic event provided the embankment foundation materials are mitigated to be able to support the vertical embankment loads as well as provide increased slope stability.

The initial reservoir configuration for the site was developed because of the anticipated cost effectiveness of such a design. **Based on the results of the analysis included herein the configuration as proposed does not successfully meet parameters that are necessary to provide for a safe and reliable embankment structure**. Based on M2C recommendations and to provide a basis of cost comparison to other project alternatives, Brelje & Race developed a modified Site 3A configuration that is presented in the 30% Conceptual Design Report that has shorter dam heights and removes and off hauls from the site weak and expansive underlying clays. The new configuration would likely perform more satisfactorily than the initial configuration analyzed but would likely accrue significant development cost increases. The alternative ponds with a lowered bottom elevation will likely be founded on the relatively more competent serpentinite bedrock materials which will settle significantly less and are better as a foundation material for the proposed embankments. The new embankments if founded on serpentinite bedrock will also likely yield the minimum safety factors required for long term static (>1.5) and short term earthquake (>1.15) loading conditions.

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# 6.0 SITE 3C-2 ANALYSIS

## 6.1 SITE 3C-2 GROUND MOTION STUDY

This section presents an analysis of the stability of the site as preliminarily designed and as influenced by material properties of the underlying soils and rock, anticipated fill materials, anticipated hydrologic characteristics and influence of ground motion from known seismogenic faulting in the region. The result of this study is a design response spectrum for Site 3C-2 and evaluation of behavior and factors of safety under various conditions. The site location is:

Latitude: 38.410989 Longitude: -122.936406

The most significant seismic event to affect the project site would likely be a maximum moment magnitude Mw 7.2 earthquake from the Rodgers Creek-Healdsburg fault zone, or a moment magnitude Mw 7.9 earthquake from the SAF. The Maximum Considered Earthquake (MCE, or 2% probability of exceedance in 50 years) is considered to rupture the entire San Andreas system with a magnitude of M 7.9.

The proposed Site 3C-2 embankments are situated on a thin layer of topsoil overlying bedrock. Although seismic refraction was performed, a shear wave velocity ($S_V$) (30m) value is not recommended from this data because it is believed that the stratigraphy has distorted the $S_V$ profile. For this reason a $S_V$ (30m) value of 760 m/sec was chosen to represent the B/C (soft rock/stiff soil) ground conditions.

M2C developed a horizontal component deterministic design response spectrum (5% damped) at the 50$^{th}$ percentile for the Site 3C-2 embankments. An 84$^{th}$ percentile spectrum was not used because the reservoir does not have characteristics of a “high hazard dam” per DSOD guidelines.

The ‘Next Generation Attenuation’ relations (NGA, 2008) developed by Abrahamson and Silva, Boore and Atkinson, Campbell and Bozorgnia, and Chiou and Youngs were used to compute estimates of median sigma ground motions. These four relations were selected as they have characterized site conditions in terms of $S_v$ (30m) in a consistent manner. Additionally, several of the relations have included depth to 1 km/sec (or 2.5 km/sec) material as independent variables to more accurately characterize strong ground motions at sites with shallow depths to bedrock materials, as occurs at the Site 3C-2 embankments. The NGA relations (NGA, 2008) provide median estimates of 5% damped pseudo absolute response spectra over the period range of 0.01 sec to 10.00 sec. Plates 28 and 29 show the spectral ordinates and target response spectrum plots respectively. The estimated peak ground acceleration at the site from the MCE on the San Andreas Fault is 0.469g.

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## 6.2 SITE 3C-2 LIQUEFACTION POTENTIAL

The materials encountered and tested within this site generally consisted of one to two feet of poor top-soil overlying bedrock materials that have index properties that indicate they are not liquefiable. Liquefaction is a secondary effect of earthquake shaking that is caused by cyclic stress that increases pore-water pressure in granular soils. As a result of the increased pore-fluid pressure, the soil undergoes a substantial loss of strength and can deform and, if the soil mass is not confined, acquire mobility sufficient to permit both horizontal and vertical displacement. Soils most susceptible to liquefaction are generally saturated, loose, clean (low clay content), uniformly graded, younger than 15,000 years, fine-grained sand or silt deposits. *The site is underlain by near surface bedrock and is therefore not liquefiable.*

Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral displacement of subsurface liquefiable material or sensitive clays. This phenomenon typically occurs adjacent to free faces such as creek channels, harbors and canals. *The site is not liquefiable, and therefore lateral spreading is not a concern.* It should be noted that the possibility of ground-shaking related random ground cracking or seismically induced slope failure affecting the site and surrounding area should not be precluded.

## 6.3 SITE 3C-2 RESERVOIR MATERIAL PROPERTIES

### 6.3.1 Site 3C-2 General

The proposed reservoir and its foundation soils/bedrock materials consist primarily of five types of materials:

1. Silty Sand Colluvium Topsoil
2. Reservoir Fill (Proposed)
3. Sandstone Conglomerate Bedrock
4. Layered Sedimentary Deposits
    (1) Graywacke Bedrock
    (2) Shale Bedrock
5. Meta-shale Bedrock

The topsoil is only expected to be one to two feet across the site and is not used in analysis, nor is it considered a foundation soil. Topsoil consists of a silty sand material and organics, but its properties are not otherwise discussed in this section.

Plates 10 and 11 show the site plan and cross sections of the dam along with the subsurface stratigraphy. The reservoir fill materials are assumed to be compacted silty to sandy clays obtained from

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excavated sandstone conglomerate bedrock materials. The excavated bedrock materials when mixed and reconstituted are expected to be silty to sandy lean clays. These lean to medium plasticity soils will need to be placed at the recommended moisture content and degree of compaction as outlined in the recommendations section.

#### 6.3.1.1 Classification and Index Tests

Grain size distribution and Atterberg limits determinations were performed on numerous samples of the sandstone conglomerate bedrock and a few tests performed on the serpentinite bedrock and graywacke bedrock. Index tests were not performed on the shale or meta-shale bedrock. Index properties of the materials are summarized below:

***Table 8: Average of the Index Test Results***

| *Material Description* | *MC[3] %* | *Total Density (PCF)* | *Dry Density (PCF)* | *Void Ratio* | *Saturation (%)* | *Specific Gravity* |
|---|---|---|---|---|---|---|
| **Dam Fill Proposed[1]** | 15 | 126.5 | 110.0 | NA | NA | 2.70 |
| **Sandstone Conglomerate Bedrock[2]** | 20.0 | 125.8 | 104.1 | 0.59 | 0.89 | NA |
| **Graywacke Bedrock[2]** | 7.3 | 137.4 | 128.0 | NA | NA | NA |
| **Shale Bedrock[2]** | NA | NA | NA | NA | NA | NA |
| **Meta-shale Bedrock[2]** | NA | NA | NA | NA | NA | NA |

1. Best engineering estimate of material parameters based on anticipated excavated materials used
2. See Hoek-Brown Failure criterion development Table 9
3. Moisture Content

### 6.3.2 Shear Strength

To estimate the shear strength of native materials, two types of laboratory testing were performed:

1. Isotropically Consolidated Undrained (ICU) triaxial compression tests with pore pressure measurements.
2. Unconsolidated Undrained (UU) triaxial compression tests.

In general, the results of the ICU tests provided the most representative results and were most suitable for use in the analysis, but the UU tests were also considered. Results from the ICU tests performed

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during investigation for Site 3A were also considered in developing representative shear strength parameters for the fill for Site 3C-2.

Shear strength tests were also performed on samples of sandstone conglomerate bedrock, but due to the destruction of material structure during sampling these tests are not considered representative and the Hoek-Brown methodology is used instead for all rock stratigraphy.

#### *6.3.2.1 Effective Shear Strength Failure Envelope*

The static shear strengths of the dam fill and foundation materials were estimated using ICU triaxial compression tests with pore water pressure measurements. The effective strength failure envelopes are shown in Appendix B. The effective material strength parameters used in the analysis are shown on Table 8A below.

***Table 8A: Effective Shear Strength Parameters***

| ***Material*** | ***Moist Unit Weight*** $^{o}$ ***(pcf)*** | ***Cohesion Intercept 'c' (psf)*** | ***Friction Angle*** |
|---|---|---|---|
| Dam Fill Proposed | 26.5 | 200 | 30.0 |
| Sandstone Conglomerate Bedrock[1] | 126.0 | Hoek-Brown | |
| Graywacke Bedrock[2] | 130.0 | Hoek-Brown | |
| Shale Bedrock[2] | 130.0 | Hoek-Brown | |
| Meta-shale Bedrock[3] | 140.0 | Hoek-Brown | |

1. Due to close and variable interbedding and inadequate information for complete segregation, these materials were grouped for stability analysis purposes.
2. The unit weight was assumed for the bulk graywacke and shale material.
3. The unit weight was assumed for the meta-shale.

#### *6.3.2.2 Total Shear Strength Failure Envelope*

For the seismic (pseudo-dynamic) analysis, undrained shear strength was used. For this study, we used the static total strength parameters from the ICU tests. The total strength failure envelopes are shown in Appendix B. Clayey soils may exhibit an increase in shear strength during rapid or seismic loading cases. The phenomenon of increase in strength for clayey materials has been well documented

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by previous studies and is a generally accepted standard of practice (Duncan and Wright, 2005). Typically, a shear strength increase of 20 percent or greater can be used for earthquake loading conditions for clayey soils. In this analysis, M2C took a more conservative approach, and did not apply any shear strength increase. Stability analyses were performed using no strength increase and are presented in Section 6.4.

*Table 8B: Total Shear Strength Parameters*

| ***Material*** | ***Moist Unit Weight $\gamma_m$ (pcf)*** | ***Cohesion Intercept c (psf)*** | ***Friction Angle $\phi$ (degree)*** |
|---|---|---|---|
| Dam Fill Proposed | 126.5 | 300 | 28.0 |
| Sandstone Conglomerate Bedrock | 126 | Hoek-Brown | |
| Graywacke Bedrock[1] | 130 | Hoek-Brown | |
| Shale Bedrock[1] | 130 | Hoek-Brown | |
| Meta-shale Bedrock | 140 | Hoek-Brown | |

1. Due to close interbedding and inadequate information for complete segregation, these materials were grouped for stability analysis

#### *6.3.3 Hoek-Brown Criterion Input Parameters*

The rock mass shear strength parameters were generated using a specific method based on research by Dr. Hoek and Dr. Brown. The Hoek-Brown Strength Criterion is an empirical rock strength criterion which takes into consideration intact rock strength, as well as influence of the discontinuities within the rock mass on the strength of the mass. Because rock mass shear strength is difficult to measure in the laboratory, (i.e. large samples are difficult to sample and test and will not contain a sufficient number of discontinuities to represent the rock mass), the rock mechanics community has adopted this approach in slope stability analysis where kinematics of the rock do not dictate the rock mass stability. The generalized Hoek-Brown strength parameters were developed for the serpentinite, sandstone conglomerate, layered graywacke and shale, and meta-shale bedrock materials using our best estimates for intact rock strengths, Geological Strength Index (GSI), material composition index and rock disturbance estimates. The disturbance factor was taken to be 0.5 for material within 15 feet of the proposed earthworks, while it was 0.1 for material outside of this area. The entire sandstone conglomerate was given a disturbance factor of 0.5 due to its proximity to surface workings and associated effects from construction activity.

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Serpentinite materials were not evaluated as direct foundation materials because of low strength properties and the desire to avoid disturbing or handling asbestiform materials that occur in the rock mass in this area.

*Table 9: Generalized Hoek-Brown Strength Parameters for Bedrock*

| Material | Total Unit Weight, $\gamma_t$ (pcf) | GSI | Mi | Disturb-ance Factor | Rock Strength (psf) | Mb | s | a |
|---|---|---|---|---|---|---|---|---|
| Serpentinite Bedrock | 140.0 | 25 | 4 | 0.5 | 12,000 | 0.11 | 4.5E-5 | 0.53 |
| Sandstone Conglomerate | 126.0 | 20 | 15 | 0.5 | 30,000 | 0.33 | 2.3E-5 | 0.54 |
| Graywacke and Shale | 130.0 | 25 | 8 | 0.1 | 60,000 | 0.48 | 2E-4 | 0.53 |
| Disturbed Graywacke and Shale | 130.0 | 25 | 8 | 0.5 | 60,000 | 0.23 | 4.5E-5 | 0.53 |
| Meta-shale | 140.0 | 50 | 10 | 0.1 | 1,500,000 | 1.53 | 3.2E-3 | 0.51 |
| Disturbed Meta-shale | 140.0 | 50 | 10 | 0.5 | 1,500,000 | 0.93 | 1.3E-3 | 0.51 |

## 6.4 SITE 3C-2 STABILITY EVALUATION

### 6.4.1 General Approach to Site 3C-2 Evaluation

Conventional limit equilibrium slope stability and earthquake-induced deformation methods were used to assess the performance of the dam. Static and pseudo-dynamic stability evaluations were carried out for upstream slopes of Northern, Eastern, Southern, and Western embankments and downstream slope of the Northern embankment. Effective shear strength parameters were used in the static stability analysis. Undrained shear strength parameters, developed as described in Section 6.3.2.2, were used in the pseudo-dynamic stability analysis. All analyses assumed that the embankments would be lined with a permanent and impermeable liner system and that the liner would be properly and regularly maintained, repaired if damaged, protected from damage from UV radiation.

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Slope stability analyses were performed using the computer program Slide 6.0 (www.rockscience.com, 2008). The program uses two-dimensional limit equilibrium technique and the method of slices, and allows the user to select various methods including simplified Bishop, simplified Janbu, Spencer, Morgenstern-Price, etc., to calculate the minimum factors of safety for both circular and block failure surfaces. The results presented for the upstream and downstream cases represent the surface type yielding the least factor of safety. Slope stability analyses were performed using the Spencer Method. In the Spencer Method, compatibility of both moment and force equilibrium is enforced, which provides a higher level of reliability than methods that compute moment or force equilibrium separately. This is specifically important when performing both static and pseudo dynamic analysis on the same surfaces as was done here.

The stability analysis herein does not offer a factor of safety for failures controlled by bedding formation dips, strikes, jointing or other linear structural bedrock features because these features, where present, were inconsistent and too disturbed/distorted for repeatable and representative structural field measurements of value to be taken.

### 6.4.2 Reservoir Profile and Cross Section

Two profiles (cwere created for Site 3C-2 (Sections G-G' and H-H') and contain representative topographic and geologic information for each of the four embankments. The representative sections are shown on Plate 11. Profiles and stability analyses are shown on Plates 30, 31 and 32 for the Northern embankment, Plate 33 for the Southern embankment, Plate 34 for the Eastern embankment, and Plate 35 for the Western embankment. A water storage elevation of 877 feet amsl was used in our analysis, which is three feet below the proposed embankment crest elevation. We have assumed that the embankment performance should allow for a minimum of one foot of free-board after the stressing events (static consolidation and earthquake induced deformations).

### 6.4.3 Long-Term Static Stability

For long-term static stability, a drained analysis with assumed steady state seepage conditions was performed for both the upstream and downstream slopes of the dam. For each slope condition, both deep and shallow failure conditions were evaluated. The analyses were performed using unit weights and effective strength parameters of the materials as shown in Table 8A and 8B.

Long term analysis of Site 3C-2 assumes the reservoir is filled to an elevation of 887 feet amsl. It is assumed that the reservoir lining does not permit any leakage through the liner, and that adequate drainage will be installed to prevent water from pooling on the back side of the liner. Results of long stability analysis for Site 3C-2 are shown on Table 10 as well as on Plates 30 through 36.

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*Table 10: Site 3C-2 Long Term Static Factor of Safety*

| Failure Type | Factor of Safety |
|---|---|
| Northern Embankment Downstream (Deep) | 3.06 |
| Northern Embankment Downstream (Fill Specific) | 3.53 |
| Northern Embankment Upstream | 4.83 |
| Southern Embankment Upstream | 3.98 |
| Eastern Embankment Upstream | 4.73 |
| Western Embankment Upstream | 5.04 |

### 6.4.4 Short-Term Static Stability

A pseudo-dynamic approach was utilized for evaluation of slope stability under seismic conditions. In this approach, earthquake forces are represented by an equivalent dynamic horizontal force which consists of the estimated effective horizontal ground acceleration multiplied by the mass of the potential slide material. For earthquake loading conditions, the analyses were performed using static total strength parameters as the undrained shear strength for the materials, as shown on Table 8B.

Based on the in-situ testing, laboratory index properties and shear strength test results, the in situ materials are not anticipated to undergo strength loss during seismic loading. Typical strength increases for rapid loading can be 20 percent or greater; however, a conservative approach of no strength increase was used.

In the pseudo-dynamic stability analysis, the result is the yield acceleration, $K_y$, an effective horizontal ground acceleration at which a potential sliding surface would develop a factor of safety equal to unity. This pseudo dynamic method was used on each of the most critical failure surfaces for long term static conditions. The results are shown on Table 11 as well as on Plates 30 through 35.

*Table 11: Site 3C-2 Pseudo Dynamic Yield Coefficient, Ky*

| Failure Type | $K_y$ |
|---|---|
| Northern Embankment Downstream (Deep) | 0.53 |
| Northern Embankment Downstream (Fill Specific) | 0.93 |
| Northern Embankment Upstream | 0.75 |

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| Failure Type | $K_y$ |
|---|---|
| Southern Embankment Upstream | 0.74 |
| Eastern Embankment Upstream | *0.78* |
| Western Embankment Upstream | *0.88* |

## 6.5 SITE 3C-2 DEFORMATIONS

Earthquake-induced deformations of the embankments were estimated using the simplified procedures by Bray and Travasarou (2007); which basically use the concept of a rigid block sliding along a failure plane within an embankment slope that was originally suggested in the Newmark's Method. In the sliding block approach, a failure slope wedge is idealized as a rigid mass above a plane of sliding with the earthquake induced accelerations. If the acceleration levels for the sliding block are lower than the yield acceleration for the sliding block, no movement will occur along the sliding plane. On the other hand, if the acceleration levels are higher than the yield acceleration, there will be net movements between the sliding block and the base, i.e., displacements. The Bray and Travasarou (2007) approach is a simplified semi-empirical method to estimate deviatoric-induced slope displacements based on the results of nonlinear fully coupled stick-slip sliding block analyses using a comprehensive database of hundreds of recorded ground motions. Plate 36 shows the results of the analysis. Given the minimum yield coefficient Ky=0.526 (the downstream side of the Northern embankment), there is a 27.18% chance of zero to one centimeter of displacement, but we estimate the horizontal earthquake induced deformations to be 6.38 cm or about 2.5 inches. The resulting crest settlement is estimated to be about 70% of horizontal or about 1.75 inches.

*Table 12: Input Parameters for Site 3C-2 Deformation Analysis*

| *Input Parameters* | *Value* |
|---|---|
| Yield Coefficient (ky) | 0.526 g |
| Initial Fundamental Period (Ts) | 0.10 seconds |
| Degraded Period (1.5Ts) | 0.15 seconds |
| Moment Magnitude (Mw) | 7.9 |
| Spectral Acceleration ( Sa(1.5Ts) | 1.079 g |

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## 6.6 SITE 3C-2 SHORT-TERM RAPID DRAWDOWN STABILITY

Total shear strength parameters were used in evaluation of the rapid drawdown stability using Slide 6.0. The analysis was carried out by changing the position of the phreatic surface in the cross-section to model the change or removal of the reservoir water level. The total strength parameters were used for the rapid draw-down analysis.

For Site 3C-2 the conditions considered were where ground water levels were brought down to the base of the interior of the reservoir while maintaining a groundwater level of 865 feet amsl. Groundwater level readings were based on two temporary piezometers (B-5 and B-7), where stabilized groundwater was measured at depths of 15 and 20.5 feet bgs respectively. These readings were assumed to be representative of groundwater conditions within the proposed embankment area.

The factor of safety for the different embankment types is shown in Table 13 as well as on Plates 30 through 35. These are above a factor of safety of 1.2 which is typically the minimum required factor of safety for a rapid drawdown scenario (USACOE, Slope Stability Manual, 2003).

*Table 13: Site 3C-2 Short-Term Rapid Drawdown Stability Factor of Safety*

| Failure Type | F.S. |
|---|---|
| Northern Embankment Downstream (Deep) | 2.10 |
| Northern Embankment Upstream | 1.32 |
| Southern Embankment Upstream | 1.71 |
| Eastern Embankment Upstream | 1.25 |
| Western Embankment Upstream | 1.30 |

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## 7.0 SITE 3C-2 CONCLUSIONS

- The majority of the planned excavations can be made with conventional medium to heavy earth moving equipment. It is possible that some localized areas of hard rock may be encountered that may be more economically excavated by blasting
- No active slope instability features have been identified within the construction limits of the site.
- No known active fault traces are present at the site, thus earthquake-induced ground rupture is not anticipated.
- The material for fill construction is medium plasticity clay obtained from the on-site excavated conglomerate bedrock materials. These materials are considered suitable for use for engineered fill materials at this or other sites.
- Neither the fill material nor the underlying bedrock formation materials are expected to experience liquefaction and therefore will not undergo significant strength loss due to a seismic event.
- The embankment deformation analysis indicates less than approximately 3 inches of horizontal movement and less than approximately 2 inches of crest settlement for the postulated strong ground shaking. These estimates are considered negligible.
- The proposed reservoir should meet the minimum safety factors required for long term static (>1.5) and short term earthquake (>1.15) loading conditions, based on the index properties from the geotechnical testing and office analysis (7.9 magnitude MCE from the controlling fault, the SAF).
- If groundwater sub drains and the reservoir lining are regularly inspected and maintained they should not pose long term maintenance problems.

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# 8.0 SITE 3C-2 RECOMMENDATIONS

## 8.1 Clearing and Grubbing

Areas to be graded including a zone extending a distance of 5 feet beyond the toe of exterior dike slopes should be cleared of debris and vegetation and the topsoil stripped so there is minimal remaining organic matter. The stripped topsoil should be removed from the construction area and stockpiled for potential reuse as topsoil. We anticipate topsoil excavation to a depth of up to 24 inches (an average of 18 inches of topsoil was found in the test pits) within the reservoir embankment area. Tree roots exceeding 2 inches in diameter should be removed. Voids created from tree stump removal or other clearing and grubbing activity should be backfilled and compacted with approved fill material as directed by the geotechnical engineer.

Areas to be cleared and grubbed include but are not limited to the embankment foundation excavations, pond bottom and side-slopes and the roadway areas.

## 8.2 Keyways

Keyways extending at least 3 feet into firm and stable soil or rock should be constructed at the toe of embankment fills. The keyways should be a minimum of 15 feet wide. The toe of the fill should extend to an imaginary 1:1 plane (45 degrees) projected from the outside base of the keyway back toward the fill. Fill over existing slopes should be placed on firm and unyielding soils or rock. Prior to constructing fills, an engineering geologist or geotechnical engineer should observe keyway excavations.

Material exposed by the excavation within the prism of the constructed dike and within pond perimeter access road (other than in the keyways), should be scarified to at least 6 inches, moisture conditioned to 1 to 3 percent above the optimum moisture content and compacted to a minimum of 90 percent relative compaction[4] or as determined by the geotechnical engineer. The geotechnical engineer should observe and approve all excavations.

## 8.3 Material Quality

In general, all excavated materials other than topsoil will be suitable for reuse as compacted fill (such as for embankments) provided organic materials and rock larger than 6 inches in diameter are re-

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[4] *Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material determined in accordance with ASTM D1557-12. Optimum moisture is the water content (percentage by dry weight) corresponding to the maximum dry density.*

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moved. All material used for construction of the levee should be free of organic material and other deleterious matter and should have a plasticity index of between 8 and 20. On-site excavated materials satisfying the above criteria, other than topsoil and strippings, are expected to be suitable for use as engineered fill if blended suitably prior to placement. All fill materials should be approved by the geotechnical engineer prior to use.

Fill material should conform to the criteria on Table 14 below:

*Table 14: Fill Material Recommended Gradation*

| Sieve Size | Percent Passing |
|---|---|
| 6-inch | 100 |
| 4-inch | 90-100 |
| No. 200 | 10-60 |
| Liquid Limit* | 40 Maximum |
| Plasticity Index* | 15 Maximum |

(*ASTM D 4318-12, wet preparation method)

Larger rock sizes may be stockpiled for later use as slope protection, landscaping, etc.

## 8.4 Fill Placement and Compaction

Approved, on-site and imported fill materials shall be placed in layers (lifts) of no more than 8 inches in uncompacted thickness. Each layer should be spread evenly and be thoroughly mixed during placement to ensure uniformity.

Fill materials should be compacted at a moisture content of 1 to 3 percent over the optimum moisture content. If the moisture content of fill material is lower than this range, water should be added to ensure thorough bonding during the compaction process. If the moisture content is above this range, the fill material should be reworked and aerated until the moisture content is as specified.

Material obtained from excavations near or below the groundwater level may be too wet to achieve the specified compaction and may require aeration by disking, scarifying, and/or spreading in layers to lower the moisture content. Drier and wetter fill materials may also be blended to achieve the specified moisture content criteria.

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After each layer has been moisture conditioned and placed, it should be compacted to a minimum of 90 percent relative compaction per ASTM 1557. All paved road aggregate base materials should be compacted to 95 percent relative compaction.

Earth moving and grading operations should be graded to prevent water from entering excavated areas. Ponded water should be promptly removed.

## *8.5 Cut and Fill Slopes*

Permanent interior embankment cut slopes should be trimmed of organic matter and should be no steeper than two horizontal to one vertical (2 : 1). Only the interior embankment fill slopes may be graded to no steeper than 2 : 1. Exterior embankment cut and fill slopes should be graded to two and one half horizontal to one vertical (2.5 : 1) and compacted using a sheeps foot compactor or other suitable equipment. Compaction operations should continue until the slope surfaces meet the specified compaction requirements and must be approved by the geotechnical engineer. Alternatively, slopes may be overbuilt and cut back to the specified grades. To reduce erosion potential, unprotected slopes should **not** be left through a winter season without providing for control of erosion. Only cut and fill slopes may be steepened, depending on their configuration, as observed during construction and must be approved by the geotechnical engineer. Slopes should be seeded with appropriate fast-growing ground cover to help reduce erosion.

## *8.6 Finished Grading*

Finished fill surfaces should be true to line and grade, and should present a smooth, firm and unyielding surface. Interior cut slopes should be constructed as directed by the manufacturer or installer of the proposed high density polyethylene or polypropylene lining system.

## *8.7 Liner*

The manufacturer or installer of the high density polyethylene or polypropylene reservoir lining system will typically have specific earthwork preparation specifications. These should be reviewed by the engineering geologist and geotechnical engineer so that the specifications are consistent with the final design drawings, specifications and bid package. We understand that the reservoir will have a floating cover that will provide protection from UV radiation. If the lining is exposed to UV radiation, it should be UV radiation resistant.

## *8.8 Surface Drainage*

Surface water runoff should be intercepted and diverted away from the site by v-ditches or compacted berms. Concentrated flows should be captured in lined ditches or closed pipes and properly discharged

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into the natural drainage. Energy dissipaters should be provided at drainage outlets and terminal pipes to prevent concentrated flows from causing erosion.

## 8.9 Subsurface Drainage

Free water was encountered in the borings; therefore, it is possible that seepage zones within the bedrock may be encountered. Depending on the volume of seepage encountered, permanent sub drains may be required. Locations and design details of sub drains can best be determined by the engineering geologist / geotechnical engineer during excavation and fill placement operations. Perforated under drain pipes, drains, toe-drains and terminal pipes should be designed to collect, convey and discharge subsurface flow adequately.

Several options are available for drainage piping and backfill and should be approved by the geotechnical engineer prior to use. If drain rock is used, it should conform to the requirements on Table 15 below:

*Table 15: Drain Rock Recommended Gradation*

| Sieve Size | Percentage Passing |
|---|---|
| 1 | 100 |
| 1 ½ | 95-100 |
| ¾ | 50-100 |
| 3/8 | 0-5 |

The permeable material should have a durability index of no less than 40 (ASTM D5313-12). Gravels and sand from the site are not suitable for this use. If a sub drainage line with collector pipes is used to drain the back side of the lined reservoir then the sub drains should be sufficiently sized and spaced to prevent groundwater from collecting and pressurizing the reservoir lining system in the case when the reservoir is drained. Groundwater level readings were based on two temporary piezometers (B-5 and B-7), where stabilized groundwater was measured at depths of 15 and 20.5 feet bgs respectively.

In-depth analysis and recommendation regarding the design of a sub-drainage system are beyond the scope of this report.

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## 8.10 Bored Outlet Line and Drain Line

A bored drain and a separate inlet/outlet pipe are planned on the west side of Site 3C-2 to relieve water from the reservoir lining system. This may be accomplished by a variety of methods including guided or unguided (straight bore) horizontal drilling, pipe jacking or ramming. If an annulus is produced from the installation method (such as by horizontal drilling) then it should be grouted to prevent water from flowing around the drainage pipe and eroding the soil confining the drainage pipe. The drain bore may encounter asbestiform-bearing serpentinite soils during installation, therefore all cuttings from the installation should be drummed and properly characterized for asbestiform materials and disposed of per regulatory requirements, or may be used onsite as long as they are not exposed upon completion.

## 8.11 Monitoring and Surveillance

Monitoring and surveillance should be performed for the reservoir both during, after (0-5 years) and on a long term basis (>5 years) after construction. Monitoring and surveillance provides a good understanding of the performance of the reservoir and will reduce the potential for incidents or failures. Typical instrumentation may include piezometers, measurement of seepage, under drain flows, inclinometers and survey monuments. The geotechnical engineer should inspect the embankments on a regular basis, particularly during and after construction. A more detailed monitoring and surveillance program should be developed during the design stage.

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# 9.0 ADDITIONAL SERVICES AND LIMITATIONS

## 9.1 ADDITIONAL SERVICES

This report was prepared for use in the final design of the project. Supplemental design recommendations and possible additional studies may be needed once the type and location of retaining structures are established. In addition, the review of plans and specifications and the field observations and testing by M2C during construction are an integral part of our conclusions and recommendations. If M2C is not retained for these services, the client will be assuming our responsibility for any potential claims that may arise during or after construction. The required tests, observations, and consultations by M2C during construction includes, but is not limited to:

- Review of plans and specifications and continued consultations during development;
- Observation of all earthwork from site clearing and stripping, through final grading and utility trench backfill;
- Construction observation and density testing of fill material and subgrade preparation; and
- Observation of foundation excavations and foundation and embankment construction.
- During and long term post construction monitoring of the embankments and associated instrumentation.

The geotechnical team on site during construction activities should consist of an experienced Geotechnical Engineer and sufficient engineering technician support to observe earthwork operations in each area of construction activity.

## 9.2 LIMITATIONS

The services provided under this contract as described in this report include professional opinions and judgments based on the data collected. These services have been performed according to generally accepted geotechnical engineering practices that existed in the project area at the time the report was written. The standard of care is time dependent. This report is issued with the understanding that the owner chooses the level of risk they wish to bear by the expenditures involved with the construction alternatives and scheduling that is chosen. In addition, the owner/developer should understand that there are risks of earth movement and property damage inherent in land development. We are unable to eliminate all risks; therefore, no expressed or implied warranty is made.

This report is issued for the sole use of our client. Others who seek to rely on the contents of this report have a duty to determine the adequacy of the report for the time, location and intended use.

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The conclusions and recommendations of this report are for the proposed development described in the text of this report. The conclusions and recommendations in this report are invalid if:

- The structures are substantially relocated;
- The report is used for adjacent or other property;
- The Additional Services section of this report is not followed;
- Changes of grades and/or groundwater occur between the issuance of this report and construction; and
- Any other change is implemented which materially alters the proposed project from the time this report was prepared.

The conclusions and recommendations presented in this report are based on information obtained from the following:

- Review of available geologic and geotechnical maps and reports;
- Subsurface data from 4 borings and six CPTs for Site 3A;
- Subsurface data from 9 borings and 18 test pits for Site 3C-2;
- Geologic mapping;
- Field observations by our geotechnical & geologic engineering staff;
- The results of geotechnical laboratory testing.

The boring logs and CPT plot summaries do not provide a warranty as to the conditions which may exist at the entire site. The extent and nature of subsurface soil and groundwater variations may not become evident until construction begins. It is possible that variations in soil conditions could exist beyond the points of exploration that may require additional studies, consultation, and possible design revisions. If conditions are encountered in the field during construction, which differ from those, described in this report, our firm should be contacted immediately to provide any necessary revisions to these recommendations.

It is the addressees and/or developers responsibility to see that all parties involved the project, including the designer, contractor, and subcontractors are made aware of this report in its entirety including the Additional Services and Limitations sections.

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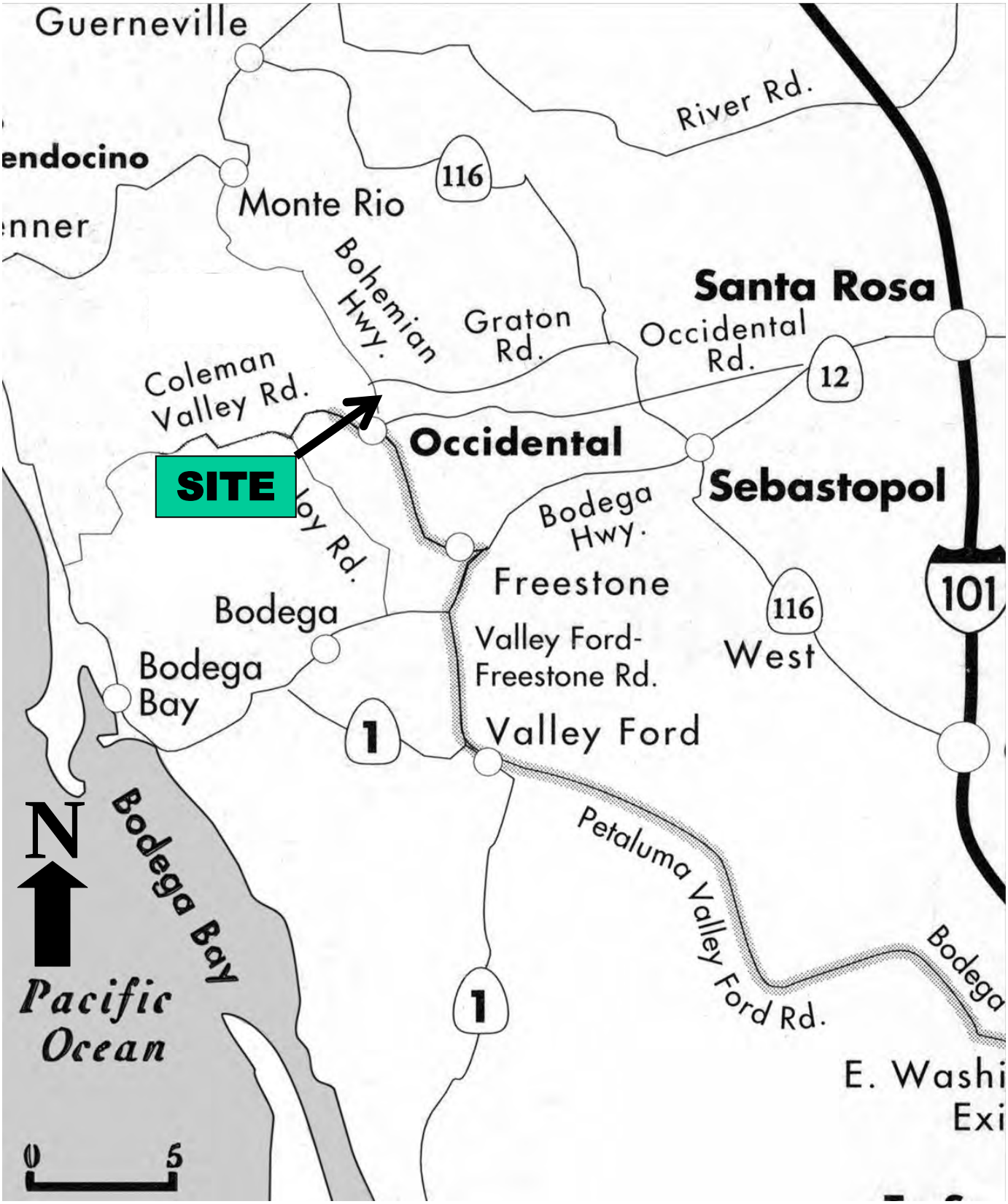
# PLATES 1 THROUGH 36

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| COMPILED BY: A. Johns | **SITE VICINITY MAP** | M2 CONSULTANTS inc | **PLATE** |
| REVISED BY: A. Johns | | | **1** |
| CHECKED BY: Y. Prashar | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | 638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | |
| DATE: Jan 18, 2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE: PLATE 1, Site Vicinity Map | | |

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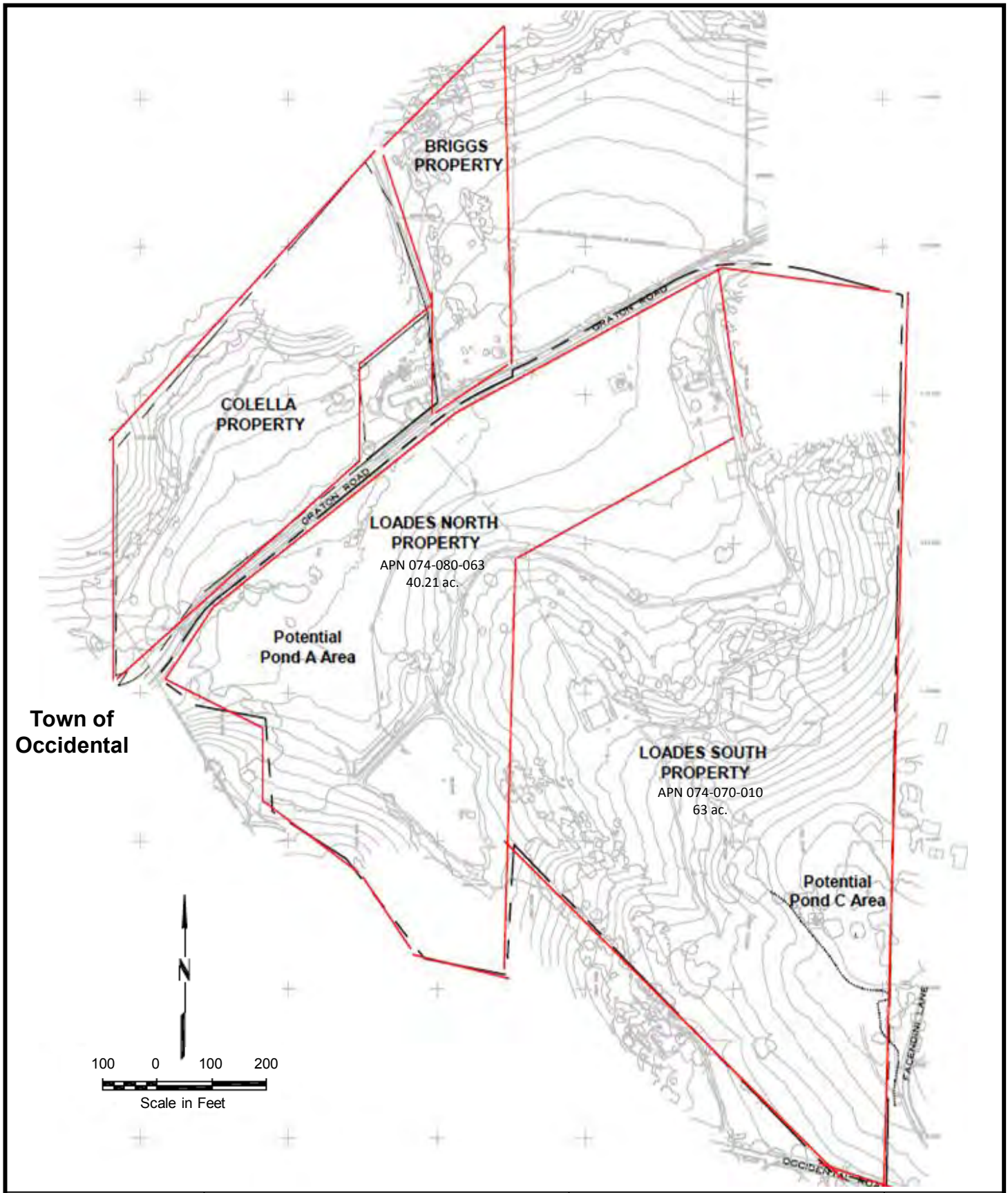


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| COMPILED BY: A. Johns | | SITE PARCEL PLAN | M2 CONSULTANTS inc | PLATE |
| REVISED BY: A. Johns | | | | 2 |
| CHECKED BY: Y. Prashar | | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | |
| DATE: Jan 18, 2013 | APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO | FILE: PLATE 2, PROJECT AREA | |

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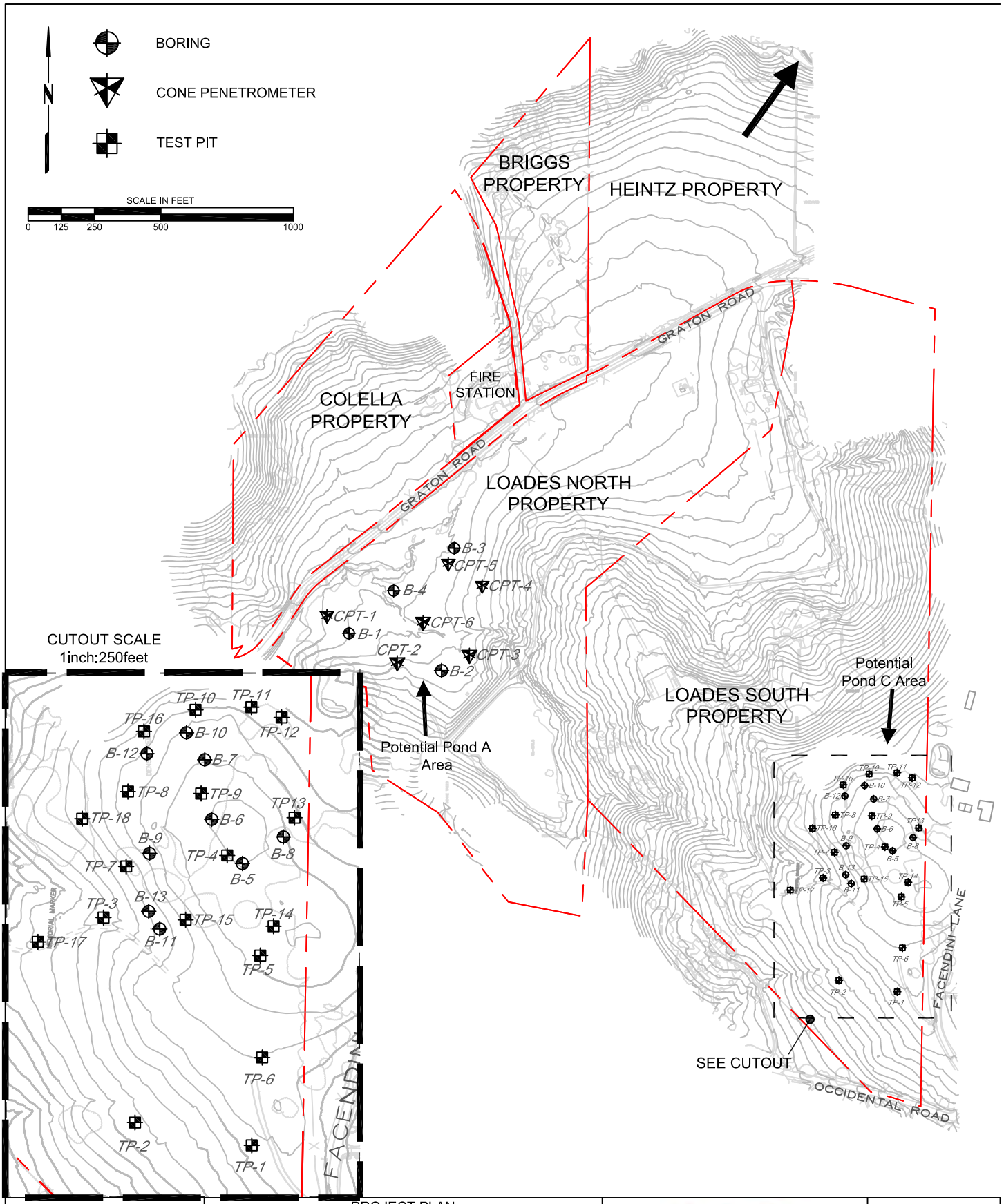


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| COMPILED BY: N. SCHEEMAKER | PROJECT PLAN<br>GEOTECHNICAL FIELD INVESTIGATION<br>BORING, CPT & TEST PIT LOCATION MAP | M2 CONSULTANTS inc<br>638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | PLATE NO.<br>3 |
| REVISED BY: N. SCHEEMAKER | | | |
| CHECKED BY: A. JOHNS | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/20/2013 / APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 / SCALE: 1 inch:500 feet | | |

## DESCRIPTION OF MAP UNITS

(See Witter and others (2006), for more information on Quaternary units).

**af** — **Artificial fill (historical)** – May be engineered and/or non-engineered.

**adf** — **Artificial dam fill (historical)** – Earth dams, rock-fill dams, and embankments constructed to impound water.

**Qhc** — **Stream channel deposits (modern to latest Holocene)** – Fluvial deposits within active, natural stream channels composed of loose sand, silt, and gravel.

**Qhay** — **Alluvial deposits, undivided (latest Holocene)** – Fluvial sediment deposited on the modern flood plain.

**Qhty** — **Stream terrace deposits (latest Holocene)** – Stream terrace deposits of sand, silt, gravel, and minor clay. Estimated to be latest Holocene in age based on elevation and/or records of historical inundation.

**Qhb** — **Basin deposits (Holocene)** – Sediment accumulated in topographic basins from slow moving or standing water. Consist of horizontally stratified sand, silt, and clay; may be interbedded with lobes of coarser alluvial deposits.

**Qhf** — **Alluvial fan deposits (Holocene)** – Sediment deposited by streams emanating from canyons onto alluvial valley floors. Sediments are typically moderately to poorly sorted and composed of sand, gravel, silt, and occasionally clay.

**Qha** — **Alluvial deposits, undivided (Holocene)** – Alluvium deposited in fan, terrace, or basin environments that could not be readily separated for mapping. Typically, consist of poorly to moderately sorted sand, silt, and gravel that form smooth geomorphic surfaces with little to no dissection.

**Qht** — **Stream terrace deposits (Holocene)** – Moderately well sorted and bedded sand, gravel, silt, and minor clay deposited in overbank and point-bar settings along streams.

**Qf** — **Alluvial fan deposits (Holocene to latest Pleistocene)** – Moderately to poorly sorted deposits of sand, gravel, silt, and clay mapped on gently sloping, fan-shaped, relatively undissected, alluvial surfaces where age of deposits is unknown.

**Qa** — **Alluvial deposits, undivided (Holocene to latest Pleistocene)** – Sand, gravel, silt, and clay mapped in small valleys and where separate fan, basin, terrace, and active stream channel units could not be delineated at the scale of mapping.

**Qls** — **Landslide deposits (historical to Pleistocene)** – Arrows indicate direction of movement; queried where landslide existence is questionable.

**Qoa** — **Older alluvium (early to late Pleistocene)** – Undivided alluvial fan, stream terrace, and basin deposits. Gray to brown, orange- to red-weathering, poorly sorted, sand, silt, and gravel composed of Franciscan basement material with conspicuous red and green chert, and lesser volcanic clasts.

**Qot** — **Older Stream terrace deposits (early Pleistocene?)** – Yellowish-brown, orange to reddish- brown-weathered, silty sand, and gravel. Lithologically similar to Qoa, but more deeply weathered and found at higher elevations.

**Tt / Twg** — **Wilson Grove Formation (late Pliocene to late Miocene)** – Light-gray to yellow-brown marine sandstone. The sandstone is fine- to medium-grained, well-sorted, massive or thick-bedded, and locally fossiliferous or tuffaceous. Well-rounded pebbles of chert and quartz occur in scattered stringers and lenses of pebbly sandstone. Tuffaceous lenses and more persistent beds vary from less than 1 cm to over 5 m thick. Thicker beds distinguishable at map scale (Tt) vary laterally and vertically from gray tuffaceous sandstone to white water-laid tuff and yellow to gray pumice breccia. Portions appear lithologically similar to the informally named Roblar tuff (Sarna-Wojcicki, 1992) that is found interbedded with the Wilson Grove and Petaluma formations farther west, and most reliably dated by Ar/Ar at 6.26 Ma (McLaughlin and others, 2008); however, definitive correlations within the study area remain to be established.

**Tpm** — **Middle Petaluma Formation (late Miocene)** – Fluvial deposits previously mapped as part of the Wilson Grove Formation (Travis, 1952; Blake and others, 1971); assigned to Tpm based on lithology and stratigraphic relationships described by Allen (2008). Light-gray, yellowish-gray, brown-weathering, sandstone and conglomerate. Conglomerate is both clast- and matrix-supported, with sub-angular to sub-rounded clasts of graywacke and other Franciscan basement rocks, with subordinate volcanic clasts.

### Franciscan Complex Coastal or Central Belt

**TKfs** — **Franciscan Complex, Coastal or Central Belt Sandstone (late Eocene to Late Cretaceous)** – Composed chiefly of broken, massive to distinctly bedded, light-gray to greenish-gray, brown- and orange-weathering sandstone. Unit also includes some argillite and shale with disrupted bedding. Sandstone is mostly feldspathic-lithic wacke with detrital biotite and muscovite (Blake and others, 2002).

### Franciscan Complex Central Belt

**KJfs** — **Franciscan graywacke and mélange (Late Cretaceous to Early Jurassic)** – Predominantly broken and locally sheared, massive to distinctly bedded, gray to green, brown- to orange-weathering, lithic wacke, and dark-gray shale. Also includes areas of mélange (see KJfm) not differentiated from the more coherent graywacke due to extensive forest cover, gradational contacts, and/or size relative to map scale.

**KJfm** — **Franciscan Complex, Central Belt mélange (Late Cretaceous to Early Jurassic)** – Tectonic mixture of penetratively sheared argillite and graywacke that forms a matrix around more coherent rock masses of varied lithology, including greenstone (gs) and serpentinite (sp) blocks large enough to be mapped separately, as well as chert, coherent graywacke, schist, and other metamorphic rocks.

**gs** — **Greenstone** – Variably altered basalt and other mafic volcanic rocks. Dark greenish-gray to black, weathered dark-brown to orange, predominantly massive, occasionally vesicular, or with pillow structure preserved.

### Great Valley Complex

**KJgv** — **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** – Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.

**Jk** — **Knoxville Formation (Late Jurassic)** – Dark-gray to black, thin-bedded shale with occasional interbeds of biotite-lithic wacke; contains only Late Jurassic fossils.

**Jsv** — **Keratophyre and quartz keratophyre tuff (Late Jurassic)** – Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).

### Coast Range Ophiolite

**sp** — **Serpentinite (Late and Middle Jurassic)** - Highly sheared, fault-bounded blocks and slivers of serpentinite and serpentinized harzburgite.

**spm** — **Serpentinite matrix mélange (Late and Middle Jurassic)** – Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.

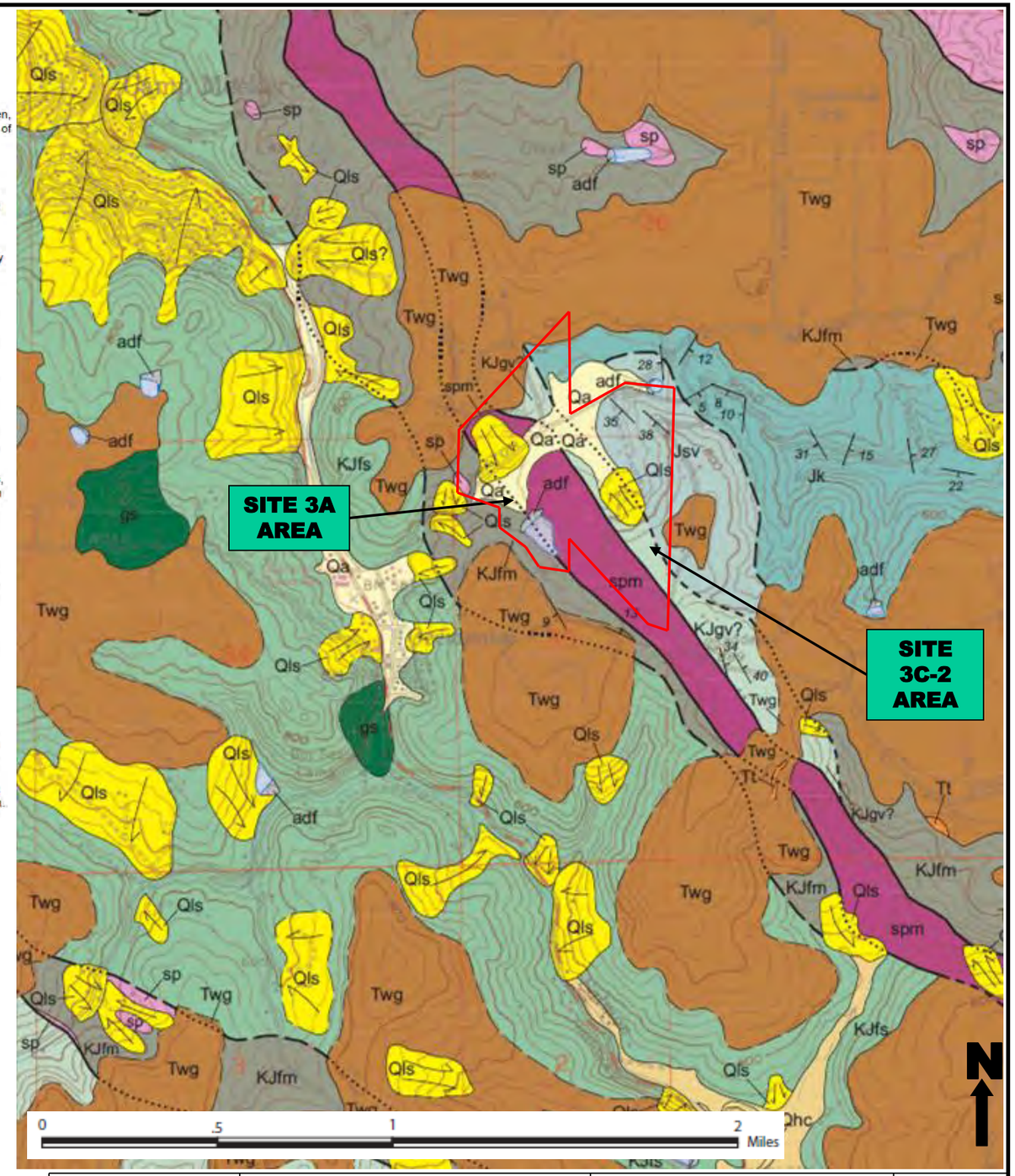
## SYMBOL EXPLANATION

Contact between map units - solid where accurately located; dashed where approximately located; short dash where inferred.

Fault - solid where accurately located; dashed where approximately located; short dash where inferred; dotted where concealed; queried where uncertain. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

Strike and dip of bedding plane

Landslide - arrows indicate principal direction of movement.




Modified from: Delattre, M.P., Koehler, R.D., 2009, Geologic Map of the Camp Meeker 7.5' Quadrangle Sonoma County, California: California Geological Survey, Scale 1:24,000.

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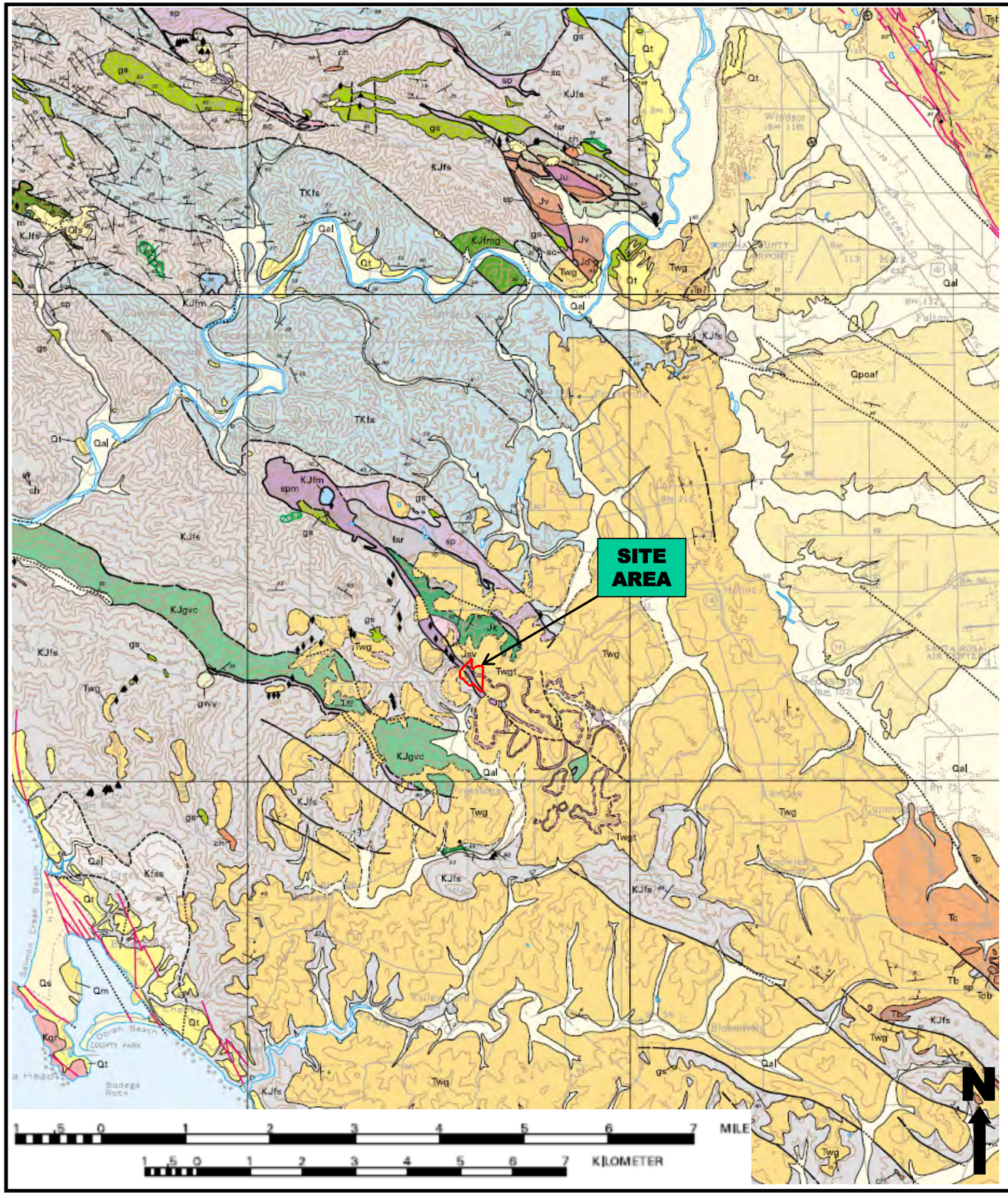
|---|---|---|---|
| COMPILED BY: A. Johns | LOCAL GEOLOGIC MAP | M2 CONSULTANTS inc | PLATE |
| REVISED BY: A. Johns | | | 4 |
| CHECKED BY: Y. Prashar | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18 San Francisco, California 94107 Ph. (415) 244-0761 | |
| DATE: Jan 18, 2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE: PLATE 4, Local Geol Map | | |

PLOTTED: 1/18/2013




## LIST OF MAP UNITS

### SURFICIAL DEPOSITS

- af — Artificial fill (Historic)
- Qal — Alluvial fan and fluvial deposits (Quaternary)
- Qm — Bay mud (Quaternary)
- Qs — Beach and dune sand (Quaternary)
- Qls — Landslide deposits (Quaternary)
- Qt — Alluvial and marine terrace deposits (Pleistocene)
- Qpoaf — Older alluvial fan deposits (Pleistocene)

### ROCKS EAST OF AND WITHIN THE SAN ANDREAS FAULT ZONE

Clear Lake Volcanics

- Qr — Rhyolite and rhyodacite (Pleistocene)
- Qob — Olivine basalt (Pleistocene)

Glen Ellen Formation (Pleistocene(?) and Pliocene)

- QTgeg — Tuffaceous member

Ohlson Ranch Formation (Pliocene)

- Tor — Sandstone
- Torc — Conglomerate
- Tls — Sedimentary rocks of Little Sulphur Creek (Pliocene and (or) Miocene)

Sonoma Volcanics (Pliocene and Miocene)

- Tsa — Andesite
- Tsb — Basalt
- Twg — Wilson Grove Formation (late Pliocene to late Miocene)
- Twgt — Tuff (late Miocene)
- Tc — Sand and gravel of Cotati (Pliocene and late Miocene)
- Tp — Petaluma Formation (Pliocene and late Miocene)
- Tb — Undivided basalt (Pliocene and Miocene)
- Tbm — Donnell Ranch volcanics of Youngman, 1989 (Miocene)

Franciscan Complex

Coastal Belt

- TKfss — Sandstone (late Eocene to Late Cretaceous, Maastrichtian)
- TKfgs — Greenstone (middle Eocene to Late Cretaceous)

Coastal or Central Belt

- TKfs — Sandstone (late Eocene to Late Cretaceous, Turonian)

Central Belt

- Kfss — Sandstone (Late Cretaceous, Turonian to Campanian)
- Kfg — Greenstone (Late(?) Cretaceous)
- Kfgwy — Sandstone (Late and Early Cretaceous, Cenomanian and (or) late Albian)
- KJfgcs — Greenstone, chert, and sandstone (Cretaceous and Jurassic)
- KJfc — Chert (Cretaceous and Jurassic)
- KJfgc — Greenstone and Chert (Cretaceous and Jurassic)
- KJfs — Graywacke and mélange (Cretaceous and Jurassic)
- fsr — Mélange
  - gs — Greenstone block
  - ch — Chert block
  - gwy — Graywacke block
  - m — High-grade metamorphic rock block
  - sp — Serpentinite block
- KJfgs — Greenstone (Jurassic)

Great Valley complex

- KJgs — Sandstone, shale, and conglomerate (Early Cretaceous and Late Jurassic)
- KJgys — Sandstone, siltstone, and shale
- KJgyc — Conglomerate
- Jk — Knoxville Formation (Late Jurassic)
- Jsv — Keratophyre and quartz keratophyre tuff (Late Jurassic)

Coast Range Ophiolite (Late and Middle Jurassic)

- Jv — Mafic and intermediate volcanic rocks
- Jd — Mafic and intermediate intrusive rocks
- Ju — Ultramafic rocks
- sp — Serpentinite
  - sc — Silica-carbonate rock
- sbm — Serpentinite matrix mélange

### ROCKS WEST OF AND WITHIN THE SAN ANDREAS FAULT ZONE

- Tsm — Sandstone and mudstone of the Fort Ross area (early Miocene)

Great Valley(?) complex

- Kg — German Rancho Formation of Wentworth and others (1998b) (Eocene and Paleocene)
- TKu — Undifferentiated German Rancho and Gualala Formations (Eocene to Late Cretaceous)

Gualala Formation of Wentworth and others (1998b) (Late Cretaceous and Paleocene?)

- Ka — Anchor Bay member
- Ks — Stewarts Point member
- Kjsb — Spilite near Black Point (Cretaceous? or Jurassic?)

Salinian complex

- Kqd — Quartz-diorite of Bodega Head (Cretaceous)

Eastern Belt

- KJfm — Metagraywacke (Cretaceous and Jurassic)
- KJfmg — Metabasalt (Cretaceous and Jurassic)

### MAP SYMBOLS

- Contact -- Depositional or intrusive contact or large mélange block edge; dashed where approximately located; dotted where concealed
- Fault -- Dashed where approximately located; small dashes where inferred; dotted where concealed; queried where location is uncertain; magenta denotes Quaternary active fault
- Strike and dip of bedding
- Strike and dip of bedding, top indicator observed
- Strike and dip of bedding, approximate
- Overturned bedding
- Crumpled bedding
- Horizontal bedding
- Vertical bedding
- Strike and dip of foliation
- Strike and dip of foliation and bedding
- Vertical foliation
- Vertical joint
- High-grade block, mappped locally
- Low-grade block, mapped locally
- Coherent greywacke within KJfs
- Foliate metabasalt

Modified from Blake, M.C. et al, 2002, Geologic map and of western Sonoma, northern most Marin, and southernmost Mendocino, counties, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-2402.

| | | | |
|---|---|---|---|
| COMPILED BY: A. Johns | REGIONAL GEOLOGIC MAP | M2 CONSULTANTS INC<br>638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | PLATE<br>5 |
| REVISED BY: A. Johns | | | |
| CHECKED BY: Y. Prashar | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | | |
| DATE: Jan 18, 2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE: PLATE 5, Reg Geol Map | | |

PLOTTED: 2013/1/18

0 20

Approximate Scale (miles)

N

Reference: Jennings, C.W., 1994, Fault Activity Map of California and Adjacent Areas: California Division of Mines and Geology, Geologic Data Map No. 6, Scale 1:750,000

**EXPLANATION**

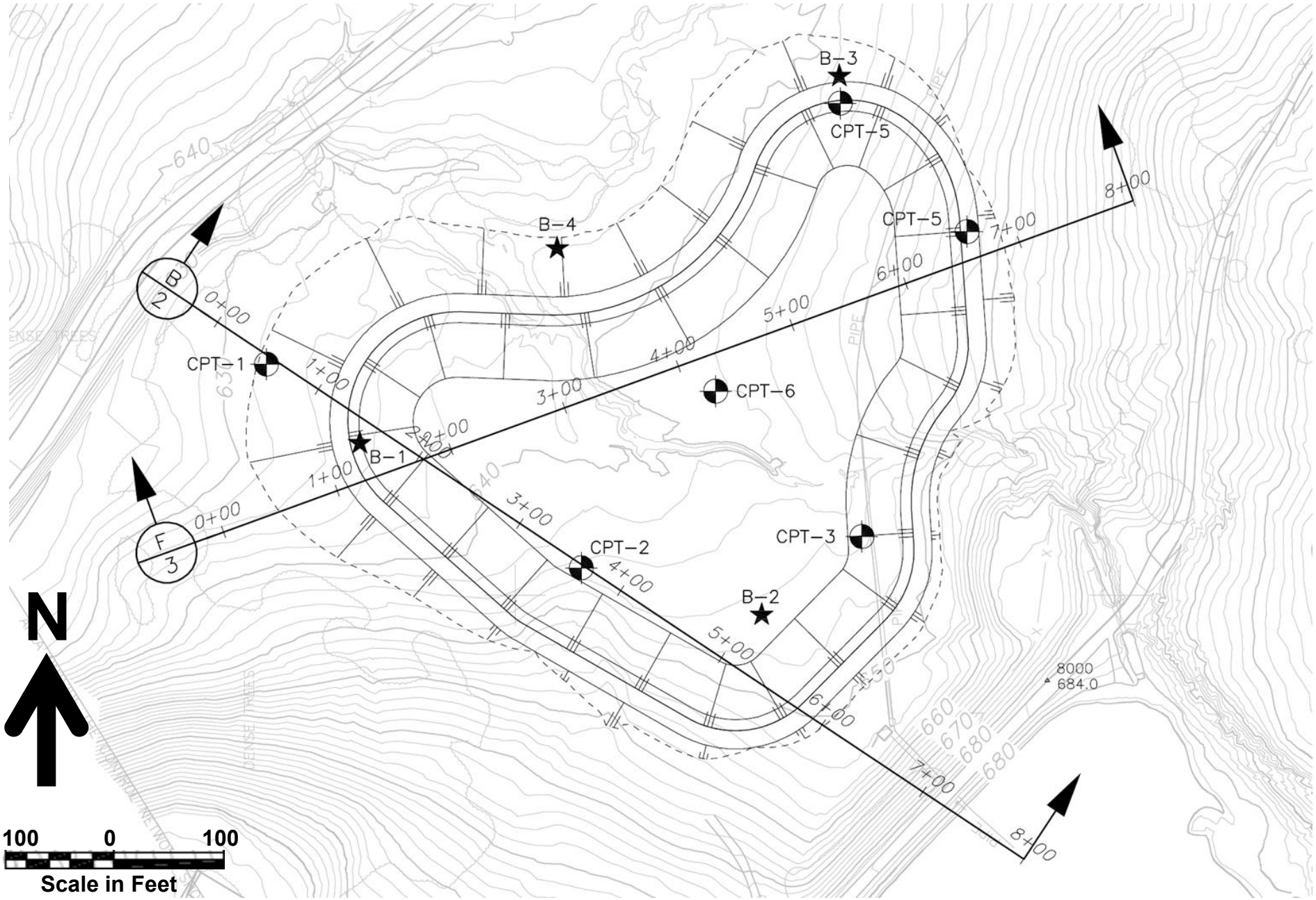
- Fault with historic movement
- Fault with movement within the last 10,000 years
- Faults with movement older than 10,000 years or undifferentiated

Original in Color

| | | | |
|---|---|---|---|
| COMPILED BY: A. Johns | **REGIONAL FAULT MAP** | M2 CONSULTANTS inc | **PLATE** |
| REVISED BY: A. Johns | | | 6 |
| CHECKED BY: Y. Prashar | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | 638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | |
| DATE: Jan 18, 2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE NAME: PLATE 6 FAULT MAP | | |

PLOTTED: 2013/1/18

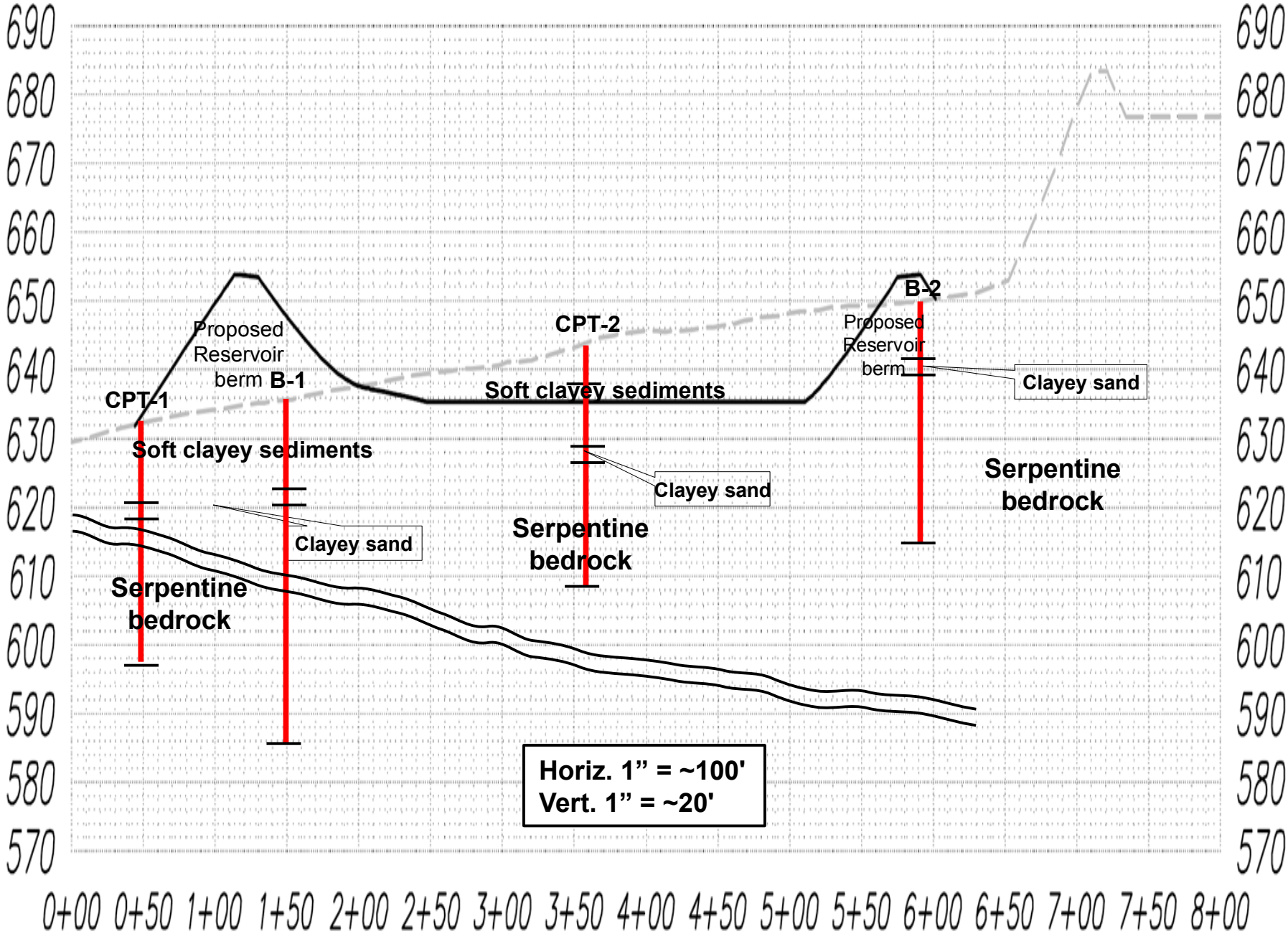
Library file: L:\2013\library\projects\011813\Fault Map.ppt




| | | | |
|---|---|---|---|
| COMPILED BY: A. Johns | **CROSS SECTIONS PLAN** | M2 CONSULTANTS inc<br>638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | **PLATE** |
| REVISED BY: A. Johns | | | |
| CHECKED BY: Y. Prashar | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT**<br>**OCCIDENTAL, CALIFORNIA** | | **7** |
| DATE: Jan 18, 2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE: PLATE 7, CROSS SECTIONS PLAN | | |

PLOTTED: 2013/1/18

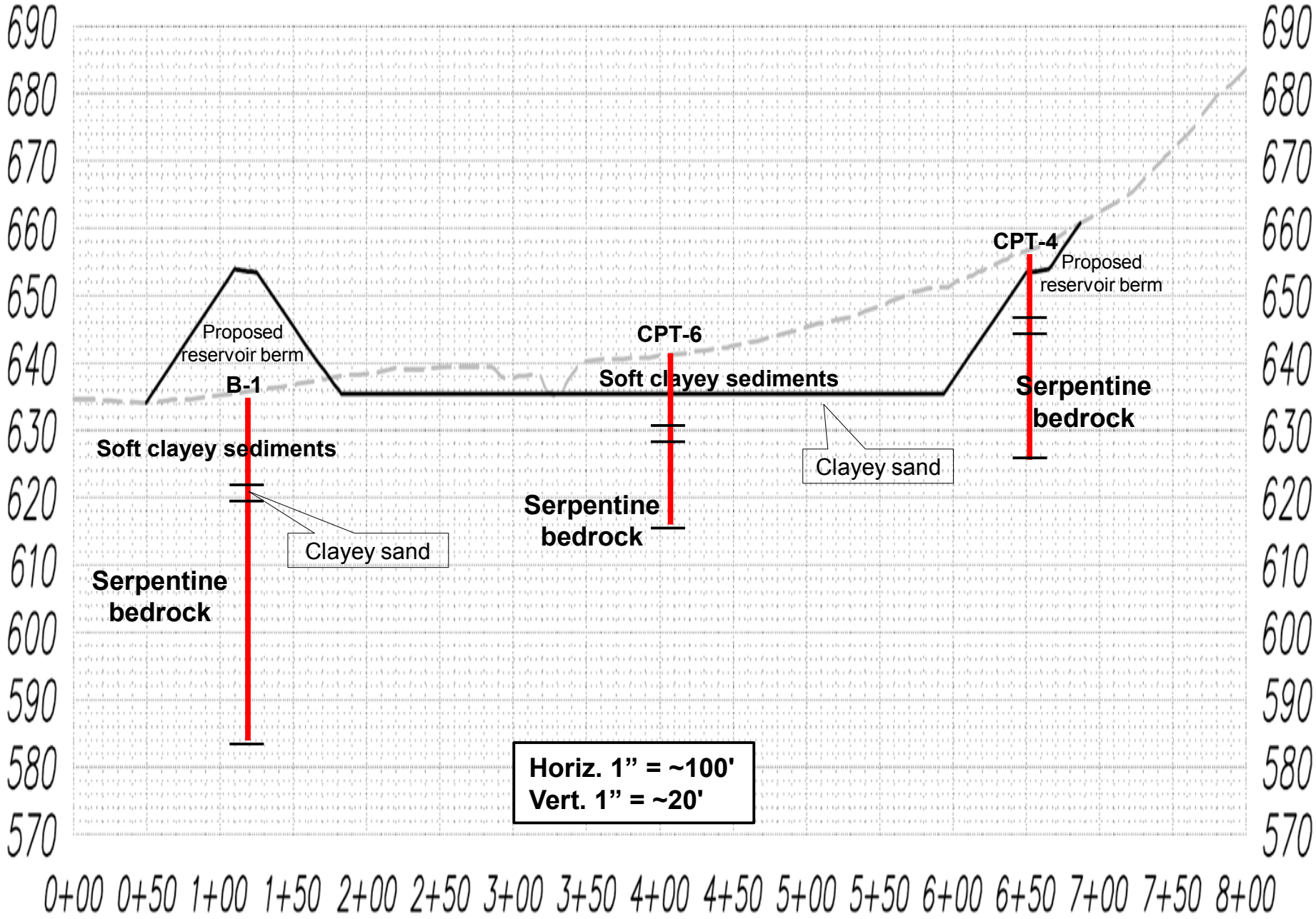
Library file: L:\2013\PLATE 7, CROSS SECTIONS PLAN.ppt




| | | | | |
|---|---|---|---|---|
| COMPILED BY: A. Johns | | SITE 3A CROSS-SECTION B | M2 CONSULTANTS inc | PLATE |
| REVISED BY: A. Johns | | | | |
| CHECKED BY: Y. Prashar | | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | 8 |
| DATE: Jan 23, 2013 | APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO | FILE: PLATE 8, CROSS-SECTION B | |

Library file: L:\2012\PLATE 8, CROSS-SECTION B.ppt

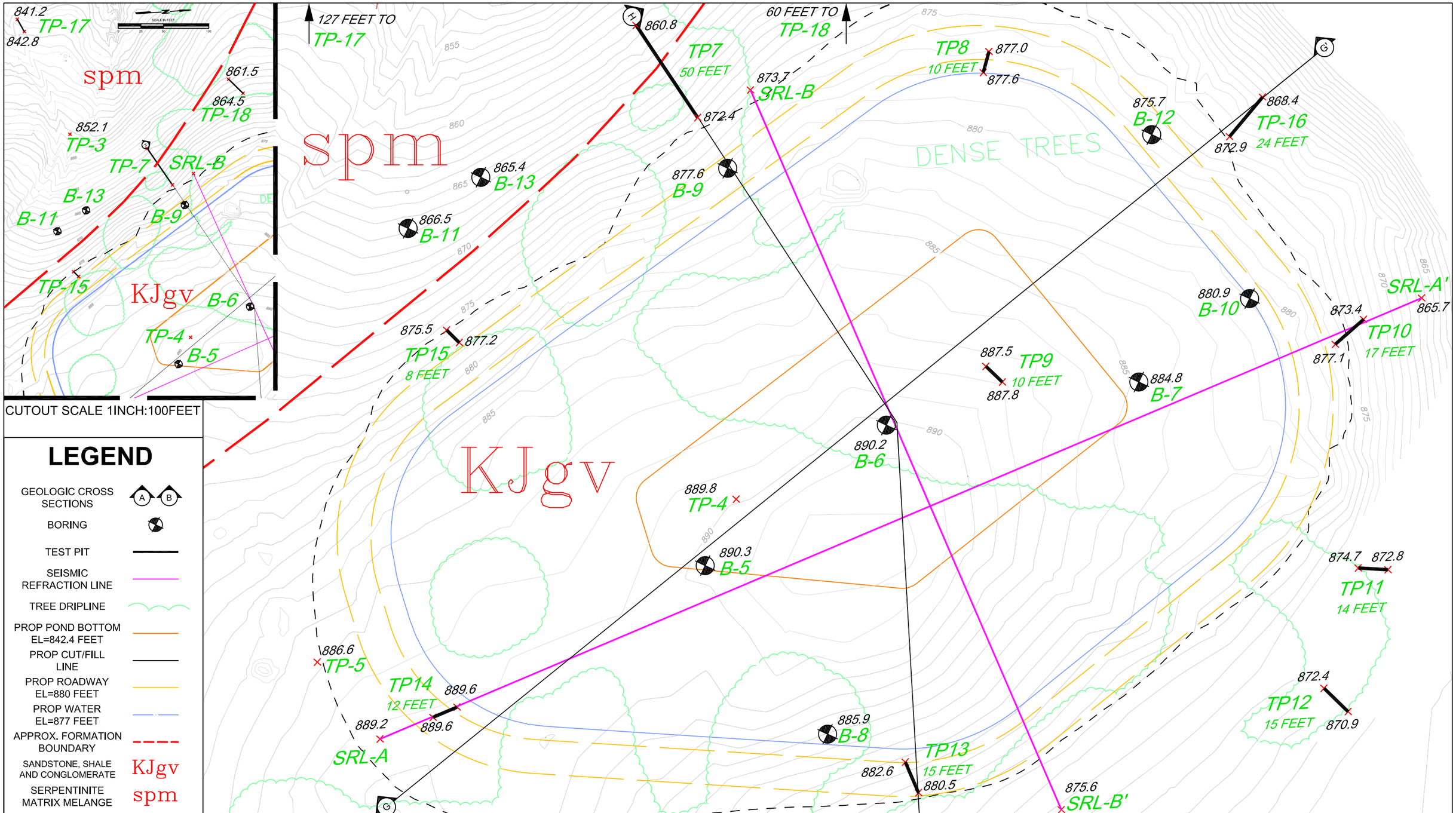
PLOTTED: 1/23/2013




| | | | |
|---|---|---|---|
| COMPILED BY: A. Johns | SITE 3A GEOLOGIC CROSS SECTION F | M2 CONSULTANTS inc | PLATE |
| REVISED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18 San Francisco, California 94107 Ph. (415) 244-0761 | 9 |
| CHECKED BY: Y. Prashar | | | |
| DATE: Jan 23, 2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE: PLATE 9, CROSS SECTION F | | |

Library file: L:\2013\PLATE 9, CROSS-SECTION F.ppt

PLOTTED: 1/23/2013




CUTOUT SCALE 1INCH:100FEET

## LEGEND

- GEOLOGIC CROSS SECTIONS (A B)
- BORING
- TEST PIT
- SEISMIC REFRACTION LINE
- TREE DRIPLINE
- PROP POND BOTTOM EL=842.4 FEET
- PROP CUT/FILL LINE
- PROP ROADWAY EL=880 FEET
- PROP WATER EL=877 FEET
- APPROX. FORMATION BOUNDARY
- SANDSTONE, SHALE AND CONGLOMERATE — KJgv
- SERPENTINITE MATRIX MELANGE — spm

| | |
|---|---|
| COMPILED BY: N. SCHEEMAKER | |
| REVISED BY: N. SCHEEMAKER | |
| CHECKED BY: A. JOHNS | |
| DATE: 01/05/2013 | APPROVED BY: Y. PRASHAR |

# SITE 3C-2 GEOLOGIC CROSS SECTION PLAN

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

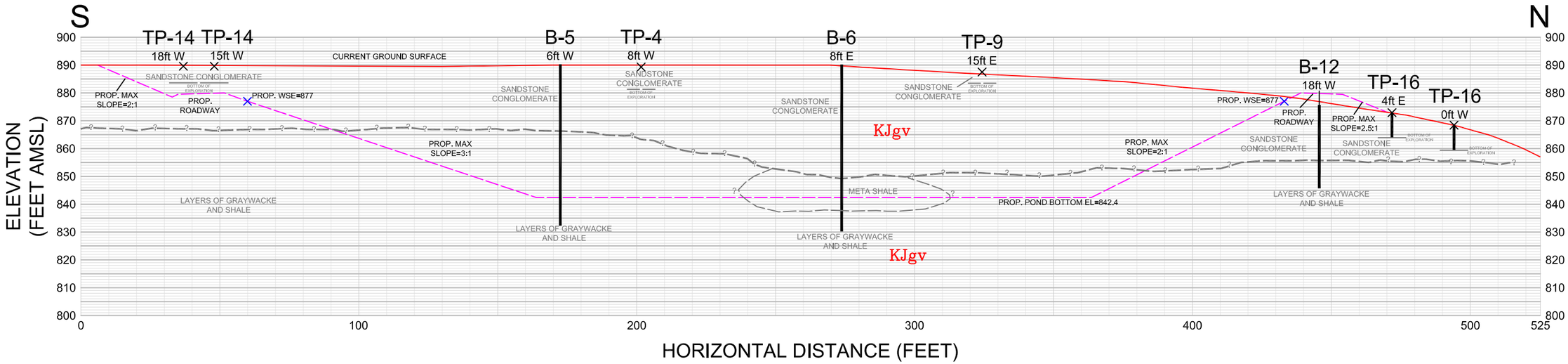
PROJECT NO: 01202 / GEO 2

SCALE: 1 inch:40 feet

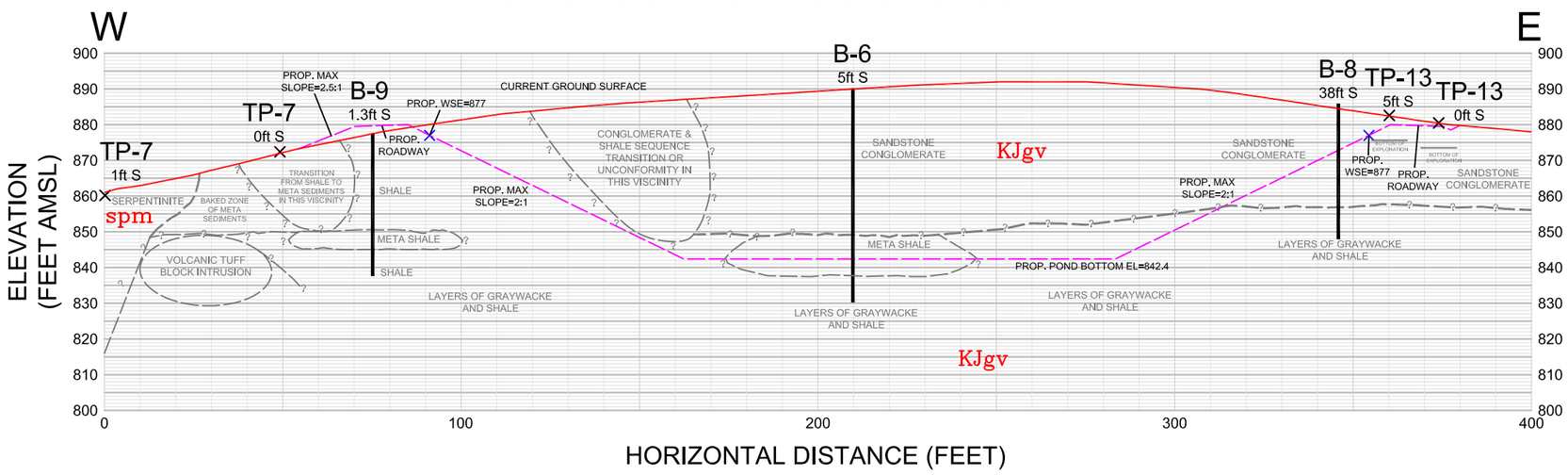
M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

PLATE NO.
10

# GEOLOGIC CROSS SECTION G-G'




# GEOLOGIC CROSS SECTION H-H'




| | |
|---|---|
| KJgv | **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lighic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite. |
| spm | **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke. |

| | |
|---|---|
| COMPILED BY: | N. SCHEEMAKER |
| REVISED BY: | N. SCHEEMAKER |
| CHECKED BY: | A. JOHNS |
| DATE: 01/05/2013 | APPROVED BY: Y. PRASHAR |

# SITE 3C-2 GEOLOGIC CROSS SECTIONS G-G' & H-H'

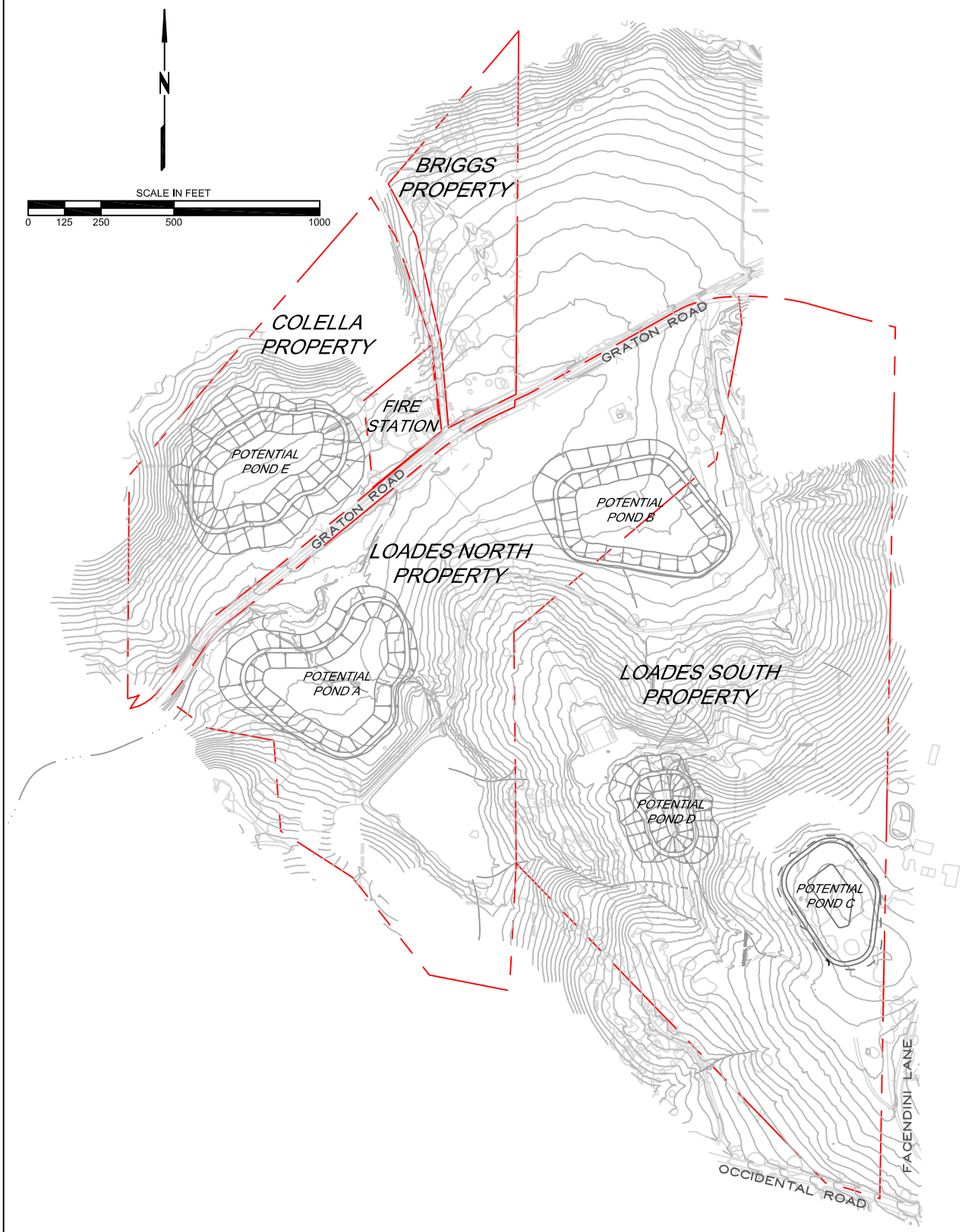
OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / GEO 2

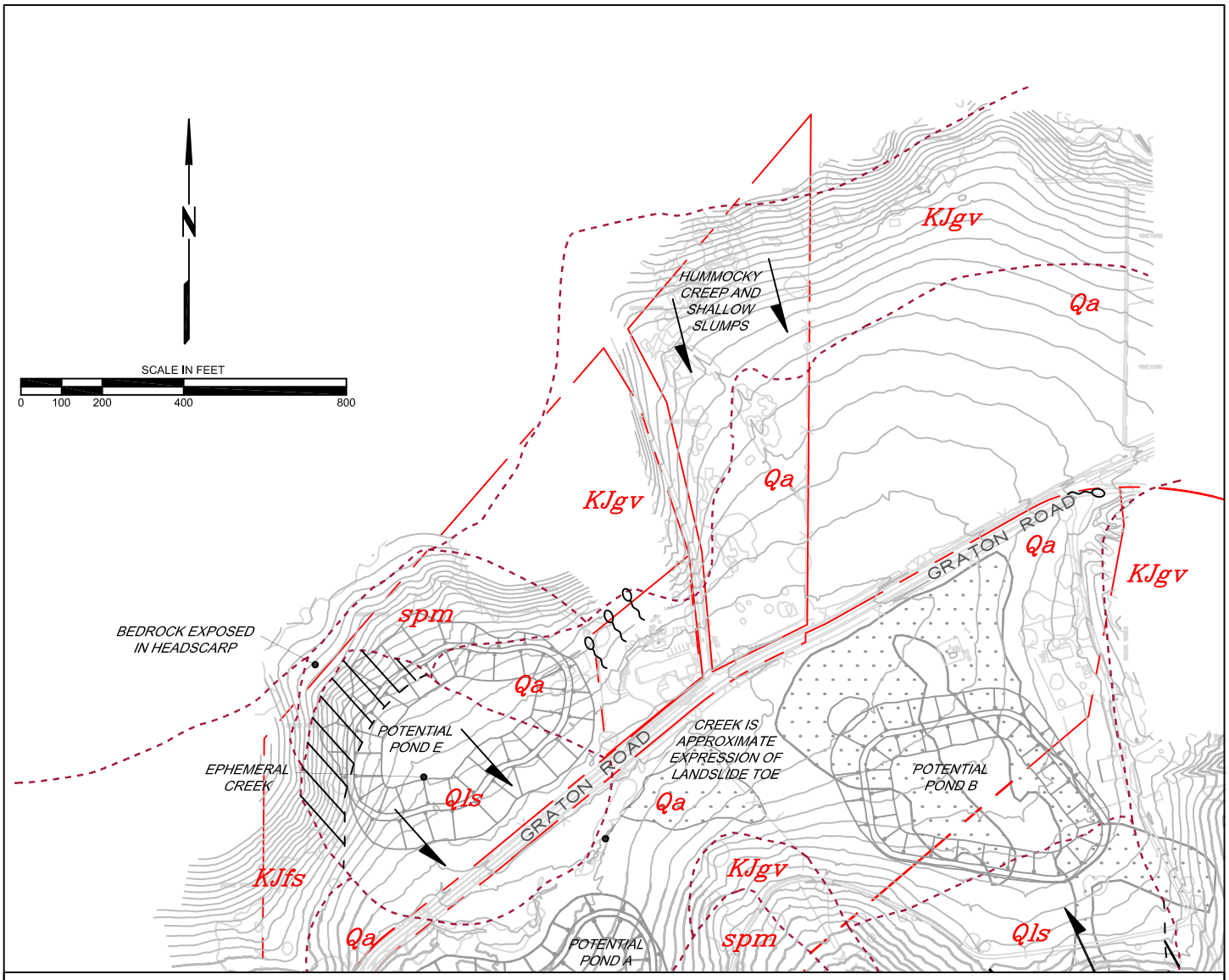
SCALE: 1 inch:40 feet

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

PLATE NO.
11




| | | | | |
|---|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | GEOLOGIC MAPS PLAN | | M2 CONSULTANTS inc | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | | 12 |
| CHECKED BY: A. JOHNS | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | | |
| DATE: 01/27/2013 / APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 | SCALE: 1 inch:500 feet | 638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | |




## SURFICIAL DEPOSITS

**Qa** — **Alluvial deposits, undivided (Holocene to latest Pleistocene)** - Sand, gravel, silt, and clay mapped in small valleys and where separate fan, basin, terrace, and active stream channel units could not be delineated at the scale of mapping.

**Qls** — **Landslide deposits (historical to Pleistocene)** - Arrows indicate direction of movement; queried where landslide existence is questionable.

## FRANCISCAN COMPLEX CENTRAL BELT

**KJfs** — **Franciscan graywacke and melange (Late Cretaceous to Early Jurassic)** - Predominantly broken and locally sheared, massive to distinctly bedded, gray to green, brown- to orange-weathering, lithic wacke, and dark-gray shale. Also includes areas of melange not differentiated from the more coherent graywacke due to extensive forest cover, gradational contacts, and/or size relative to map scale.

## GREAT VALLEY COMPLEX

**KJgv** — **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.

**Jsv** — **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).

## COAST RANGE OPHIOLITE

**spm** — **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.

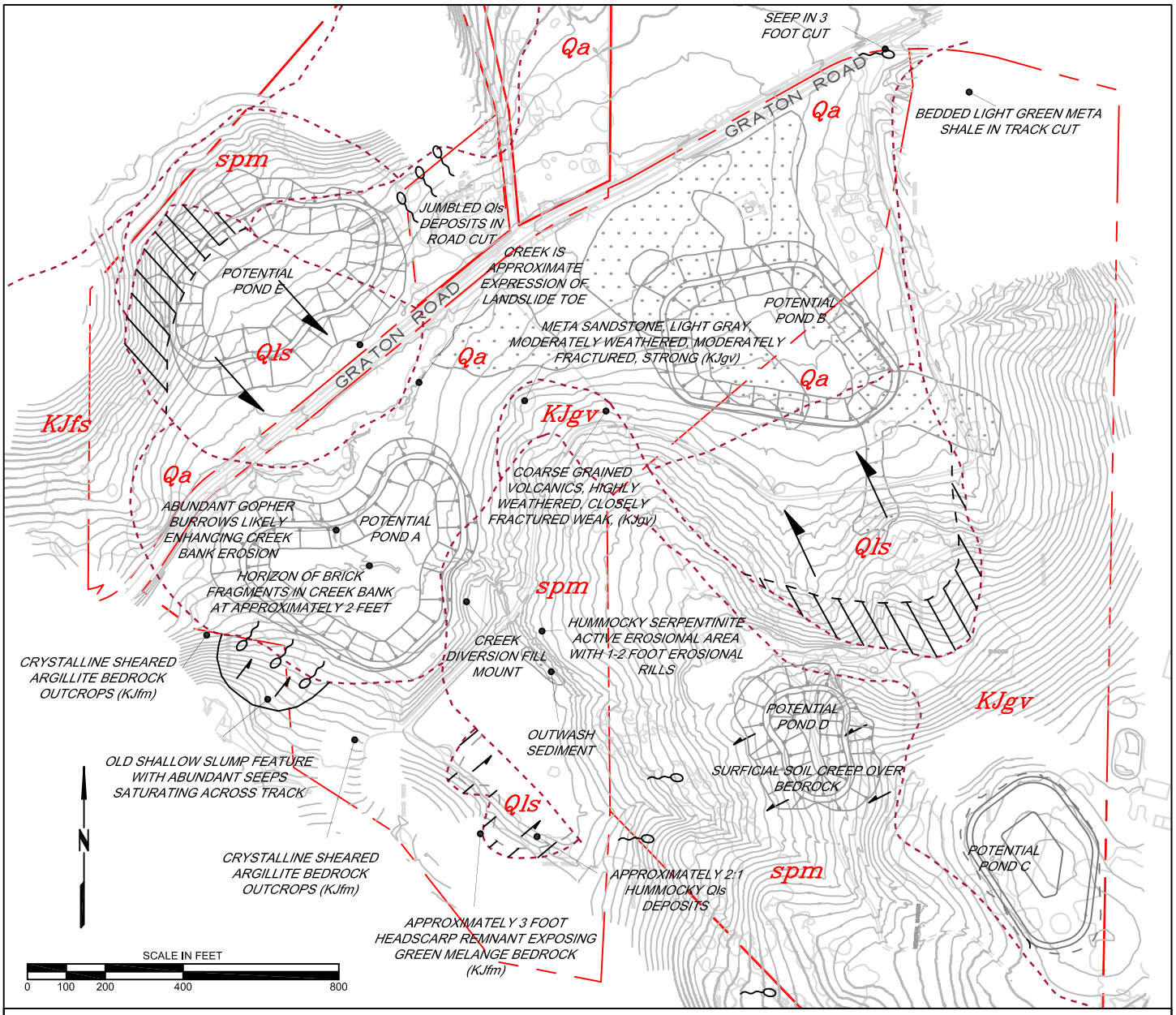
SLUMP/SLIDE DIRECTION

SEEP

APPROXIMATE FORMATION BOUNDARY

WETLANDS

| | | | |
|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | GEOLOGIC MAP OF BRIGGS & COLELLA PROPERTIES | M2 CONSULTANTS inc | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | 13 |
| CHECKED BY: A. JOHNS | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/27/2013 / APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 / SCALE: 1 inch:400 feet | 638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | |




## SURFICIAL DEPOSITS

**Qa** — **Alluvial deposits, undivided (Holocene to latest Pleistocene)** - Sand, gravel, silt, and clay mapped in small valleys and where separate fan, basin, terrace, and active stream channel units could not be delineated at the scale of mapping.

**Qls** — **Landslide deposits (historical to Pleistocene)** - Arrows indicate direction of movement; queried where landslide existence is questionable.

## FRANCISCAN COMPLEX CENTRAL BELT

**KJfs** — **Franciscan graywacke and melange (Late Cretaceous to Early Jurassic)** - Predominantly broken and locally sheared, massive to distinctly bedded, gray to green, brown- to orange-weathering, lithic wacke, and dark-gray shale. Also includes areas of melange not differentiated from the more coherent graywacke due to extensive forest cover, gradational contacts, and/or size relative to map scale.

## GREAT VALLEY COMPLEX

**KJgv** — **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.

**Jsv** — **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).

## COAST RANGE OPHIOLITE

**spm** — **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.

SLUMP/SLIDE DIRECTION

SEEP

APPROXIMATE FORMATION BOUNDARY

WETLANDS

| | | | |
|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | GEOLOGIC MAP OF LOADES NORTH PROPERTY | M2 CONSULTANTS inc | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | 14 |
| CHECKED BY: A. JOHNS | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/27/2013 / APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 / SCALE: 1 inch:400 feet | 638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | |

## OUTCROP KEY

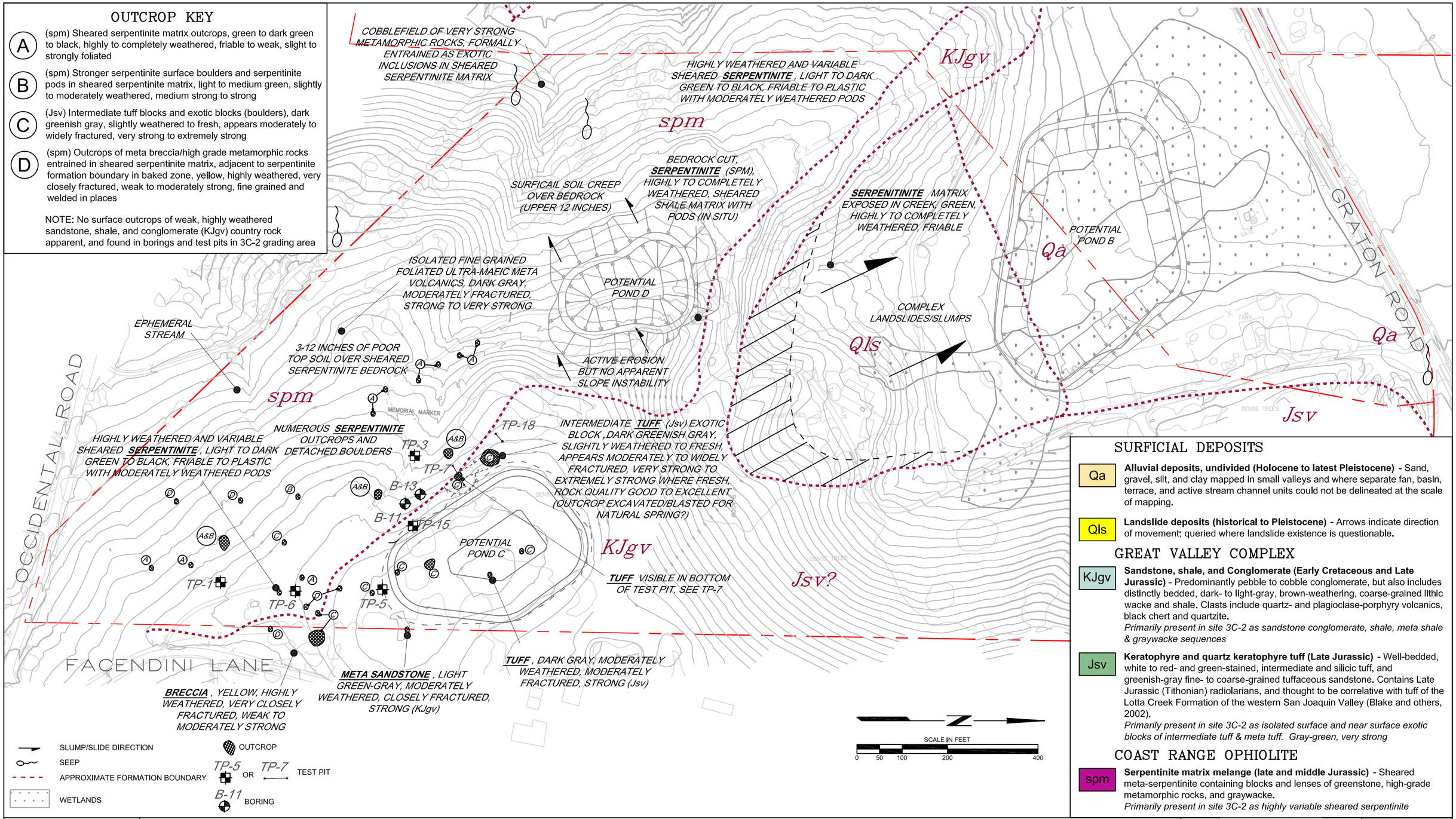
**A** (spm) Sheared serpentinite matrix outcrops, green to dark green to black, highly to completely weathered, friable to weak, slight to strongly foliated

**B** (spm) Stronger serpentinite surface boulders and serpentinite pods in sheared serpentinite matrix, light to medium green, slightly to moderately weathered, medium strong to strong

**C** (Jsv) Intermediate tuff blocks and exotic blocks (boulders), dark greenish gray, slightly weathered to fresh, appears moderately to widely fractured, very strong to extremely strong

**D** (spm) Outcrops of meta breccia/high grade metamorphic rocks entrained in sheared serpentinite matrix, adjacent to serpentinite formation boundary in baked zone, yellow, highly weathered, very closely fractured, weak to moderately strong, fine grained and welded in places

NOTE: No surface outcrops of weak, highly weathered sandstone, shale, and conglomerate (KJgv) country rock apparent, and found in borings and test pits in 3C-2 grading area




## SURFICIAL DEPOSITS

**Qa** — **Alluvial deposits, undivided (Holocene to latest Pleistocene)** - Sand, gravel, silt, and clay mapped in small valleys and where separate fan, basin, terrace, and active stream channel units could not be delineated at the scale of mapping.

**Qls** — **Landslide deposits (historical to Pleistocene)** - Arrows indicate direction of movement; queried where landslide existence is questionable.

## GREAT VALLEY COMPLEX

**KJgv** — **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.
*Primarily present in site 3C-2 as sandstone conglomerate, shale, meta shale & graywacke sequences*

**Jsv** — **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).
*Primarily present in site 3C-2 as isolated surface and near surface exotic blocks of intermediate tuff & meta tuff. Gray-green, very strong*

## COAST RANGE OPHIOLITE

**spm** — **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.
*Primarily present in site 3C-2 as highly variable sheared serpentinite*

| | |
|---|---|
| COMPILED BY: | N. SCHEEMAKER |
| REVISED BY: | N. SCHEEMAKER |
| CHECKED BY: | A. JOHNS |
| DATE: 01/27/2013 | APPROVED BY: Y. PRASHAR |

# GEOLOGIC MAP OF LOADES SOUTH PROPERTY

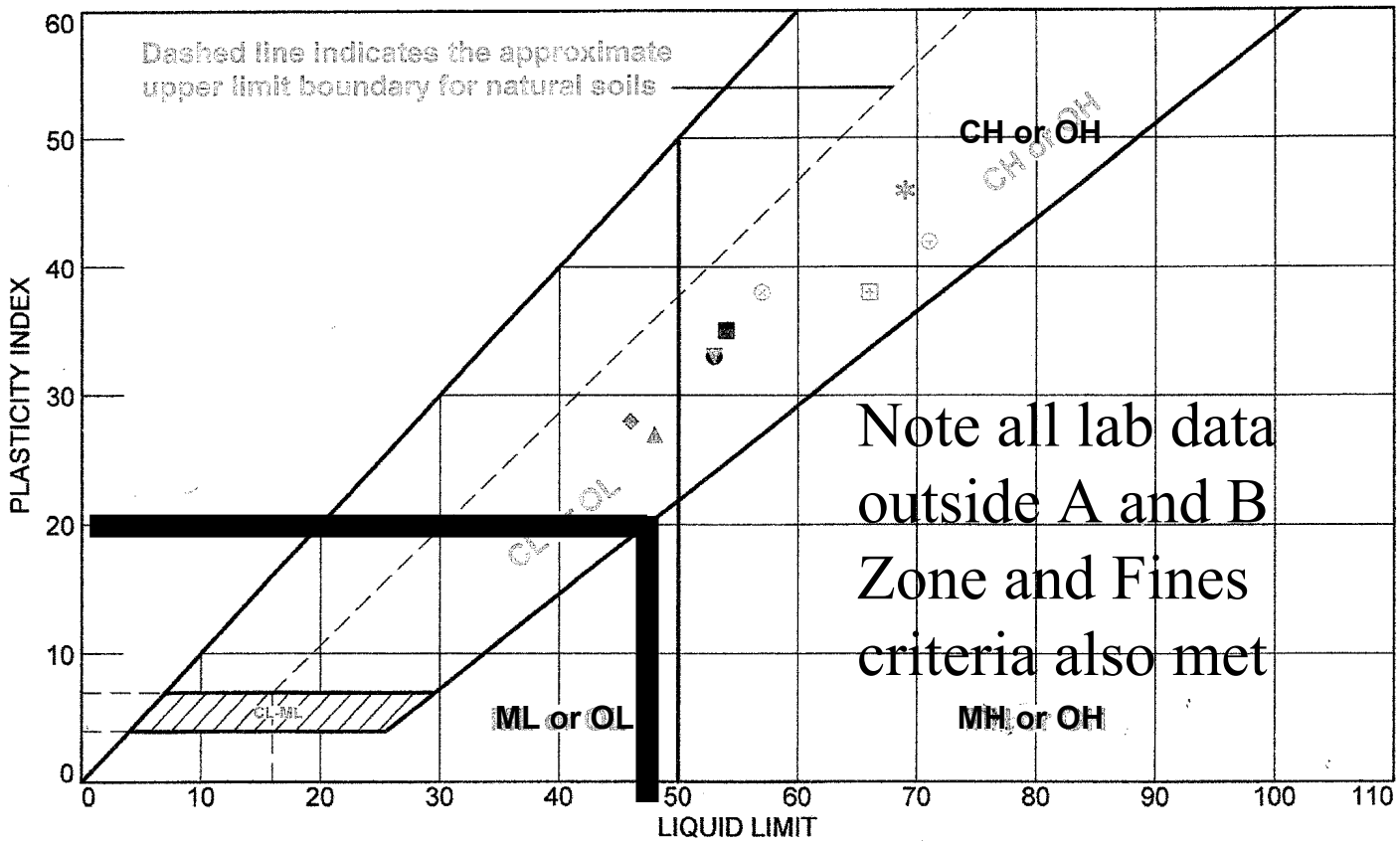
OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

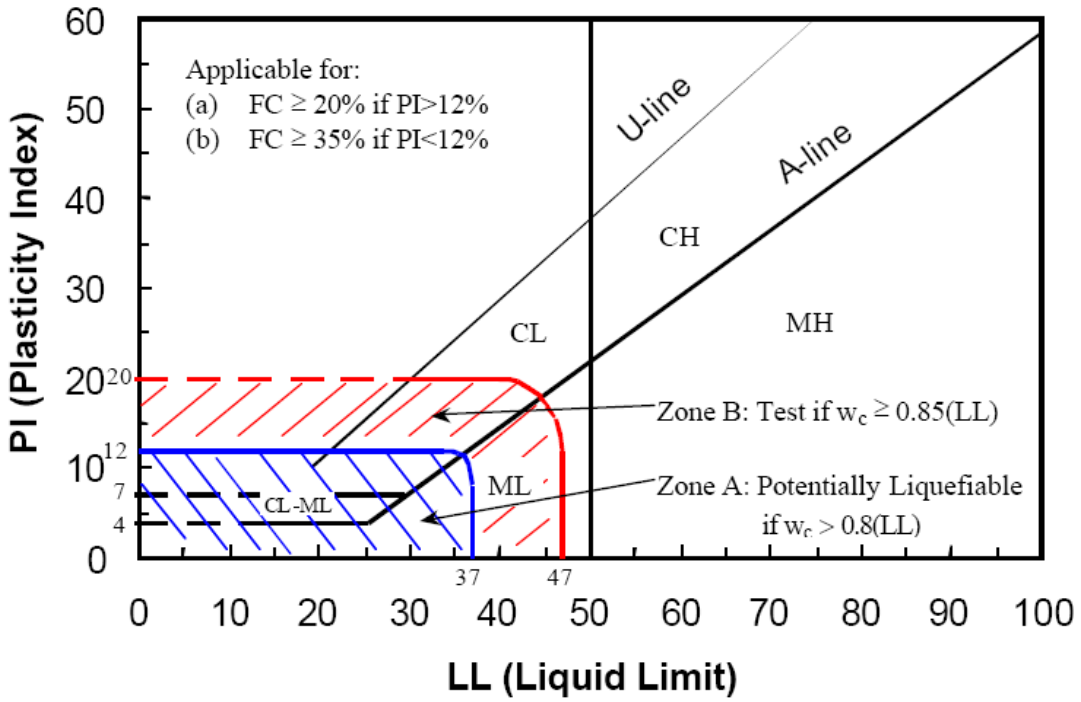
PROJECT NO: 01202 / GEO 2

SCALE: 1 inch:200 feet

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

PLATE NO. 15

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|---|---|---|---|---|
| COMPILED BY: A. Johns | **Liquefaction Potential Evaluation** | M2 CONSULTANTS inc | **PLATE** |
| REVISED BY: A. Johns | | | **16** |
| CHECKED BY: Y. Prashar | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | 638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | |
| DATE: Feb 29, 2012 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE: PLATE 1, Test Pits 1 & 2 | | |

Library file: L:\2012\library\projects\01202\SVM.ppt

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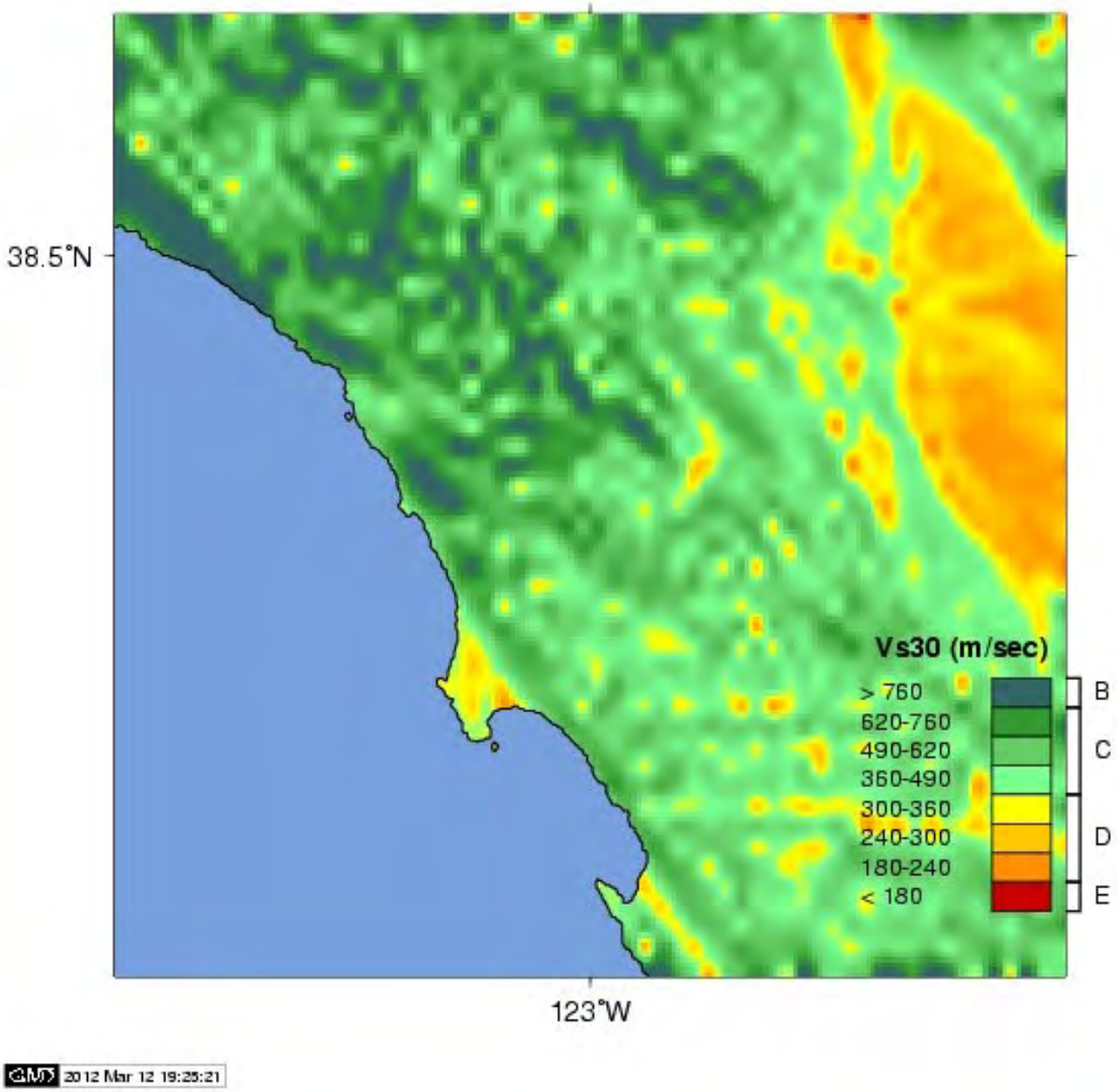



| | | | | |
|---|---|---|---|---|
| COMPILED BY: A. Johns | **Shear Wave Velocity Vs-30** | | M2 CONSULTANTS inc | **PLATE** |
| REVISED BY: A. Johns | | | | **17** |
| CHECKED BY: Y. Prashar | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | | 638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | |
| DATE: Feb 29, 2012 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO | FILE: PLATE 1, Test Pits 1 & 2 | | |

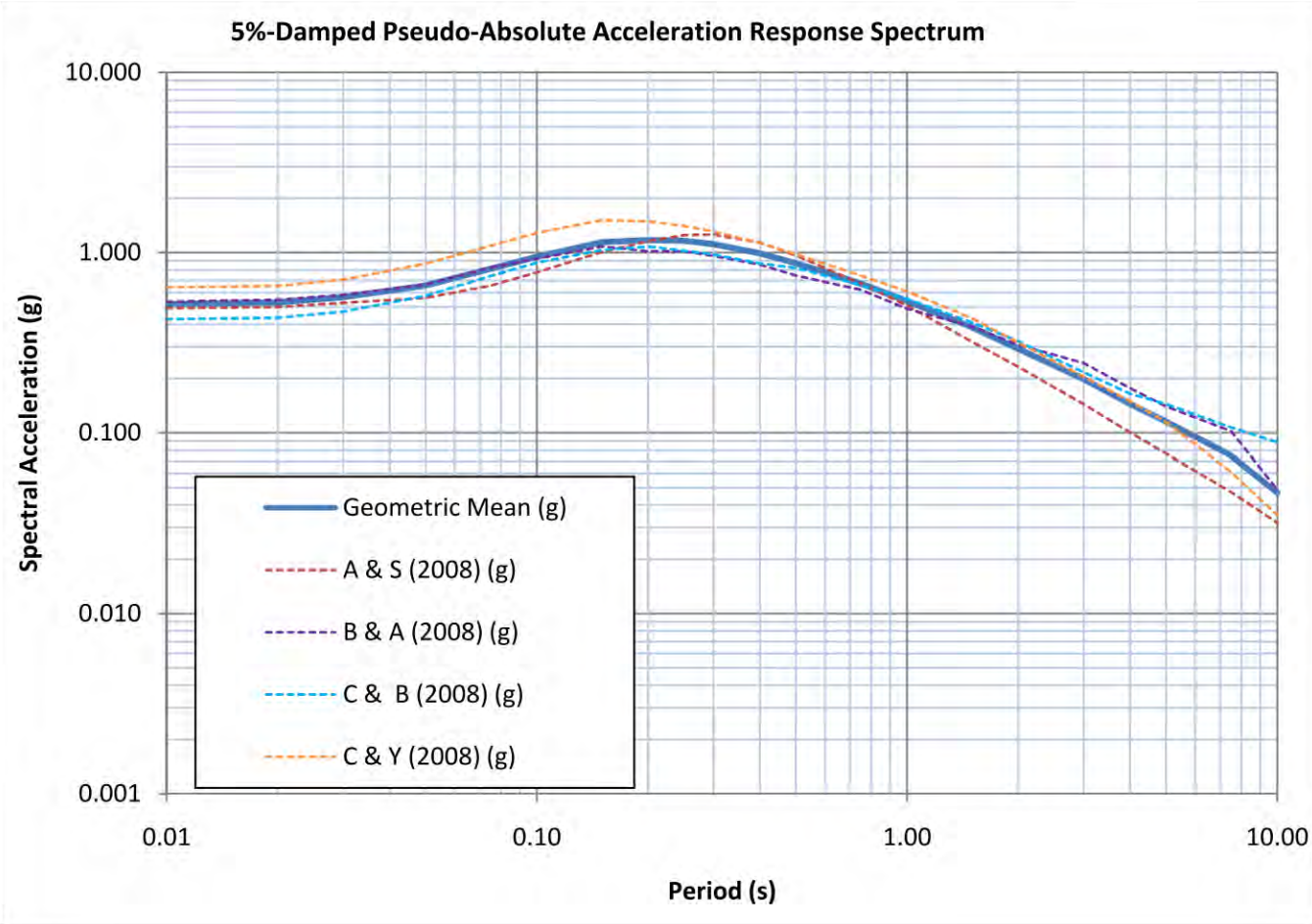
Library file: L:\2012\library\projects\01202\SVM.ppt

PLOTTED: March 21, 2012, 09:07:49 PM

| PEER Ground Motion Database Target Spectrum Report 21-Jan-2013 | | | | | |
|---|---|---|---|---|---|
| Earthquake Source Parameters | | | | Distance Parameters | |
| Mag | 7.9 | Strike Slip Faulting | | Rrup (km) | 12.16 |
| Dip (deg) | 90 | $V_{S30}$ (m/s) | 550 | Rjb (km) | 12.16 |
| Width (km) | 30 | | | Rx (km) | 12.16 |

| Period (s) | Geometric Mean (g) | A & S (2008) (g) | B & A (2008) (g) | C & B (2008) (g) | C & Y (2008) (g) |
|---|---|---|---|---|---|
| 0.01 | 0.517 | 0.490 | 0.534 | 0.426 | 0.641 |
| 0.02 | 0.527 | 0.498 | 0.547 | 0.434 | 0.653 |
| 0.03 | 0.565 | 0.525 | 0.583 | 0.470 | 0.708 |
| 0.05 | 0.656 | 0.559 | 0.663 | 0.575 | 0.867 |
| 0.08 | 0.812 | 0.655 | 0.826 | 0.740 | 1.089 |
| 0.10 | 0.948 | 0.776 | 0.923 | 0.878 | 1.285 |
| 0.15 | 1.142 | 1.008 | 1.080 | 1.033 | 1.513 |
| 0.20 | 1.172 | 1.154 | 1.015 | 1.079 | 1.491 |
| 0.25 | 1.163 | 1.250 | 1.015 | 1.024 | 1.405 |
| 0.30 | 1.113 | 1.257 | 0.957 | 0.975 | 1.309 |
| 0.40 | 0.989 | 1.137 | 0.856 | 0.869 | 1.130 |
| 0.50 | 0.872 | 0.963 | 0.745 | 0.822 | 0.979 |
| 0.75 | 0.672 | 0.673 | 0.621 | 0.659 | 0.743 |
| 1.00 | 0.537 | 0.507 | 0.487 | 0.554 | 0.609 |
| 1.50 | 0.384 | 0.319 | 0.388 | 0.409 | 0.431 |
| 2.00 | 0.291 | 0.231 | 0.307 | 0.321 | 0.316 |
| 3.00 | 0.198 | 0.144 | 0.243 | 0.216 | 0.204 |
| 4.00 | 0.145 | 0.100 | 0.176 | 0.165 | 0.150 |
| 5.00 | 0.115 | 0.077 | 0.140 | 0.144 | 0.115 |
| 7.50 | 0.075 | 0.047 | 0.102 | 0.107 | 0.061 |
| 10.00 | 0.047 | 0.032 | 0.049 | 0.088 | 0.035 |

| PEER Ground Motion Database Target Spectrum Report 21-Jan-2013 | | | | | |
|---|---|---|---|---|---|
| Earthquake Source Parameters | | | | Distance Parameters | |
| Mag | 7.9 | Strike Slip Faulting | | Rrup (km) | 12.16 |
| Dip (deg) | 90 | $V_{S30}$ (m/s) | 550 | Rjb (km) | 12.16 |
| Width (km) | 30 | | | Rx (km) | 12.16 |

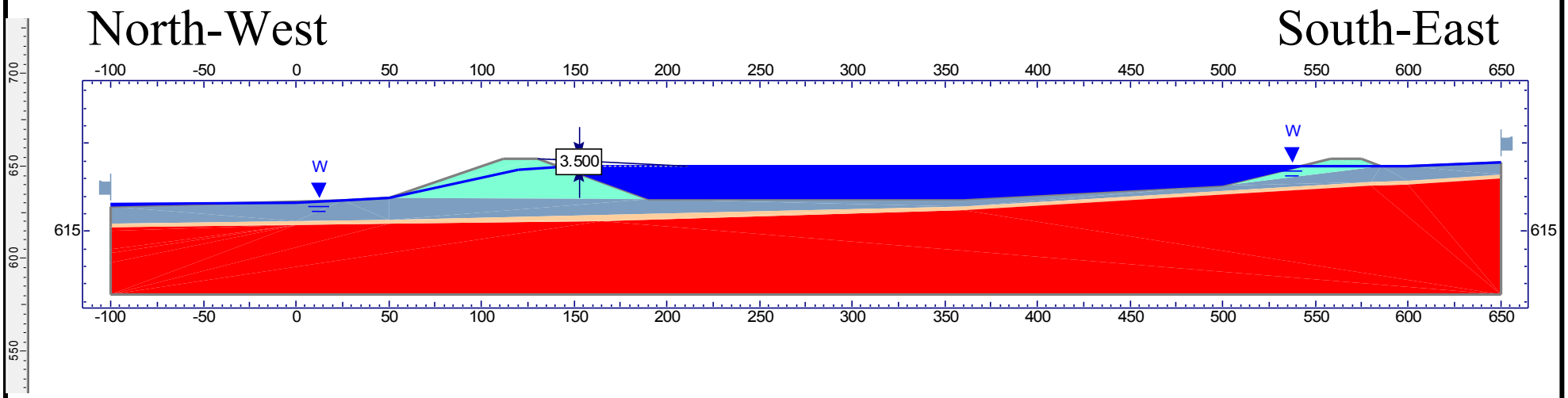



| CALCULATION OF WEIGHTED AVERAGE 2008 NGA MODELS: | | | | | |
|---|---|---|---|---|---|
| **NGA Model:** | AS08 | BA08 | CB08 | CY08 | I08 |
| **Weight:** | 0.25 | 0.25 | 0.25 | 0.25 | 0 |

| | | | | |
|---|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | | 3A SITE SPECIFIC RESPONSE SPECTRA<br>2008 NGA ATTENUATION EQUATION | M2 CONSULTANTS inc<br>638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | PLATE NO.<br>19 |
| REVISED BY: N. SCHEEMAKER | | | | |
| CHECKED BY: A. JOHNS | | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/21/2013 | APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 | | |

# Summary of all Shear Strength Parameters

| | | | | | | | Effective Shear Strength | | Total Shear Strength | |
|---|---|---|---|---|---|---|---|---|---|---|
| Boring No | Sample No. | Depth (ft) | Description | In-Situ Water Content, % | In Situ Moist Unit Weight, pcf | In Situ Dry Unit Weight, pcf | Cohesion Intercept c' (psf) | Friction Angle ϕ ' (degree) | Cohesion Intercept c (psf) | Friction Angle ϕ (degree) |
| NA | NA | NA | Dam Fill | 15.0 | 126.5 | 110.0 | 200.0 | 30.0 | 300.0 | 28.0 |
| B-1 | 1A | 1-3.5 | Soft Clay | 26.6 | 123.9 | 97.9 | 297.0 | 24.0 | 316.7 | 23.8 |
| B-2 | 3A | 5.5-8 | Clayey Sand | 36.6 | 117.2 | 85.8 | 387.0 | 25.8 | 458.9 | 16.7 |
| B-3 | 1C | 4 | Soft Clay | 29.5 | 118.8 | 91.7 | 261.2 | 23.3 | 80.1 | 25.3 |
| B-4 | 5C | 11 | Med Dense Clayey Sand | 39.1 | 112.7 | 81.0 | 114.0 | 31.1 | 229.5 | 19.3 |
| * Assumed Parameters | | | | | | | | | | |

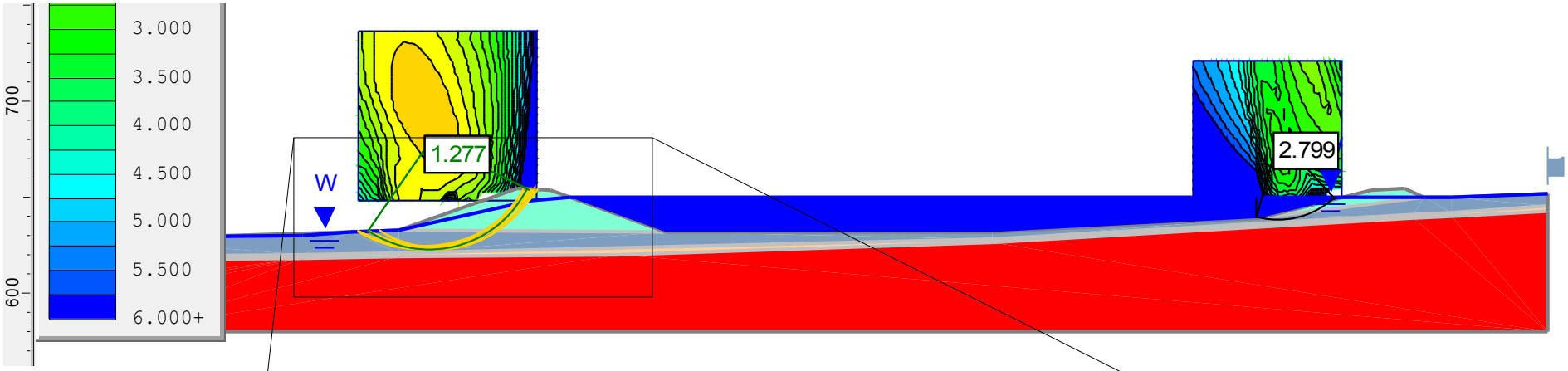



# Typical Cross-Section Used in Analysis

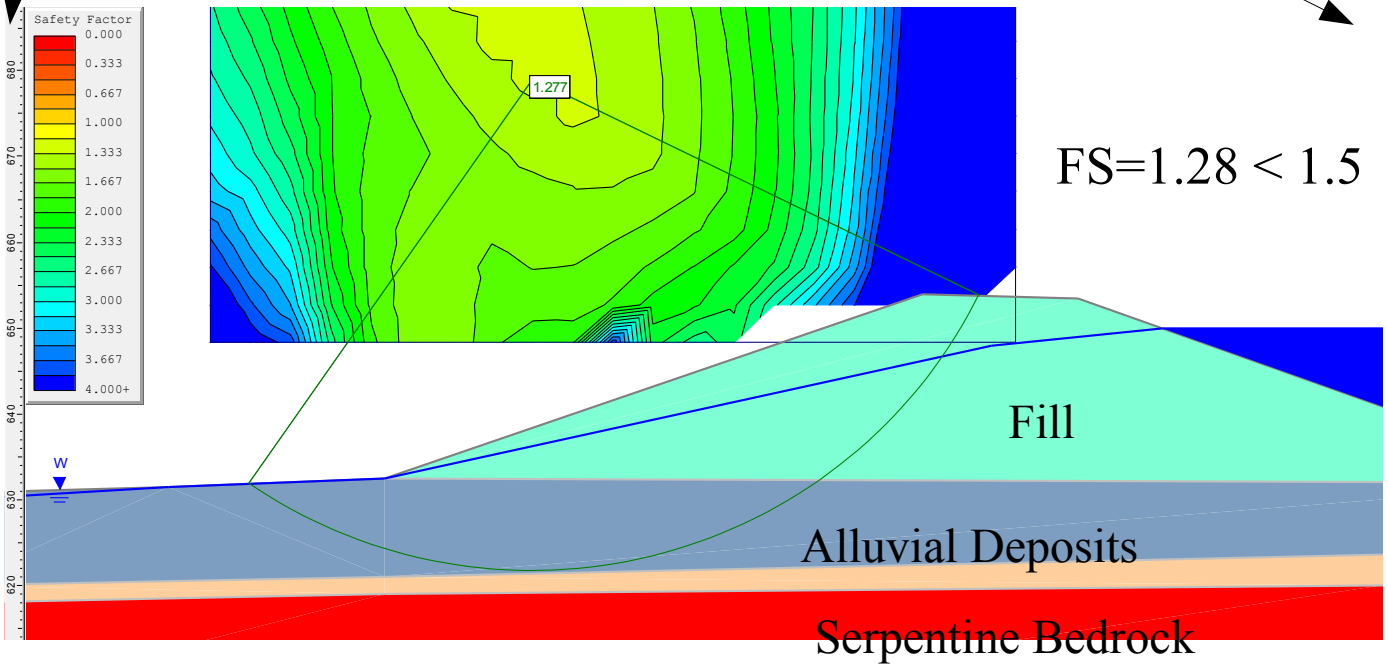
| | | | |
|---|---|---|---|
| COMPILED BY: A. Johns | **CROSS-SECTION B-B** | M2 CONSULTANTS inc | **PLATE** |
| REVISED BY: A. Johns | | | |
| CHECKED BY: Y. Prashar | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | 638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | **20** |
| DATE: MAR 11, 2012 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE: PLATE 7, CROSS-SECTION F | | |

Library file: L:\2012\PLATE 7, CROSS-SECTION F.ppt

PLOTTED: March 21, 2012, 08:49:06 PM



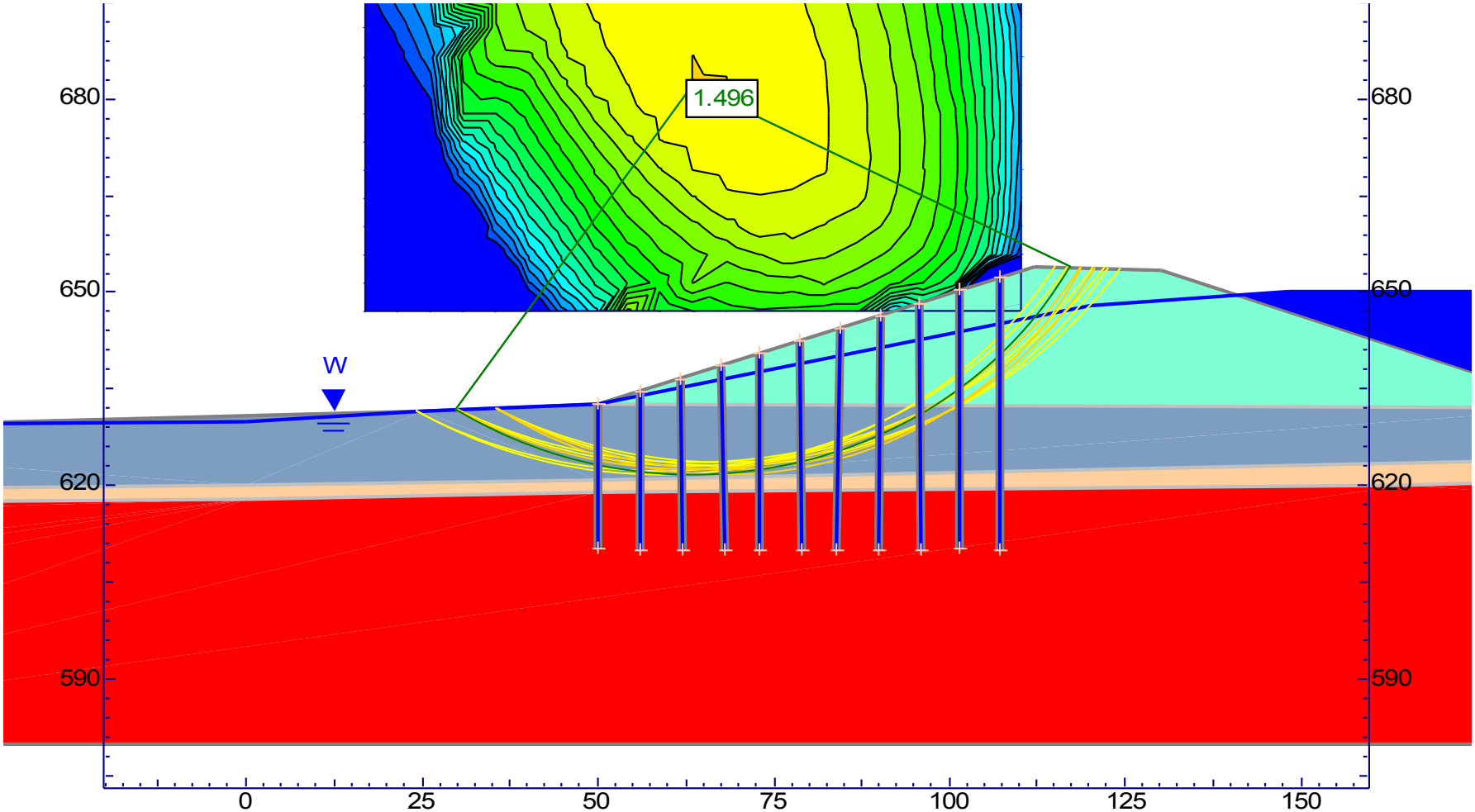

# LONG TERM STATIC CASE WITH RESERVOIR FILLED




| | | | |
|---|---|---|---|
| COMPILED BY: A. Johns | **CROSS-SECTION B-B** | M2 CONSULTANTS inc<br>638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | **PLATE** |
| REVISED BY: A. Johns | | | |
| CHECKED BY: Y. Prashar | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | | **21** |
| DATE: MAR 11, 2012 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE: PLATE 7, CROSS-SECTION F | | |

Library file: L:\2012\PLATE 7, CROSS-SECTION F.ppt

PLOTTED: March 21, 2012, 08:49:06 PM

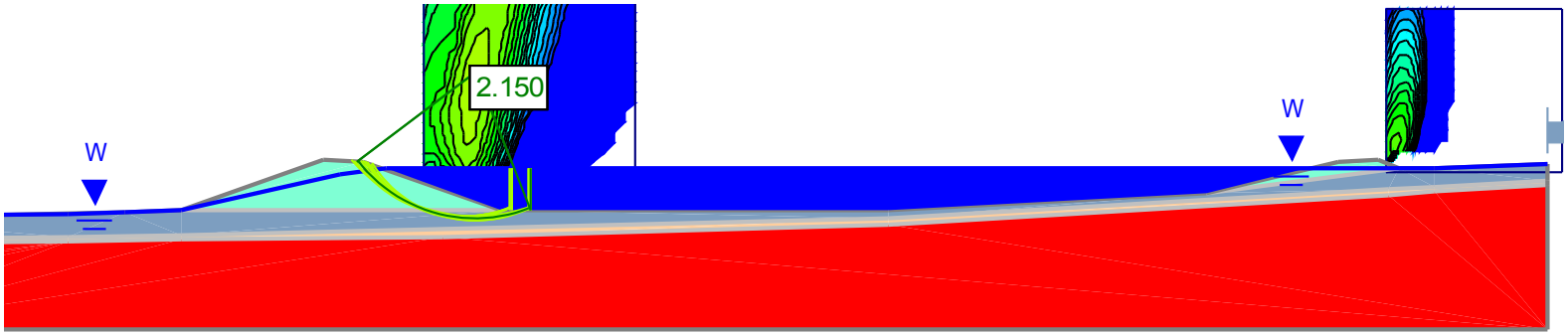



Conceptual Design:
Long Term Static Case - Reinforced with 6' X 6' grid of Rammed Aggregate Piers (www.geopiers.com) embedded into Serpentine Br.

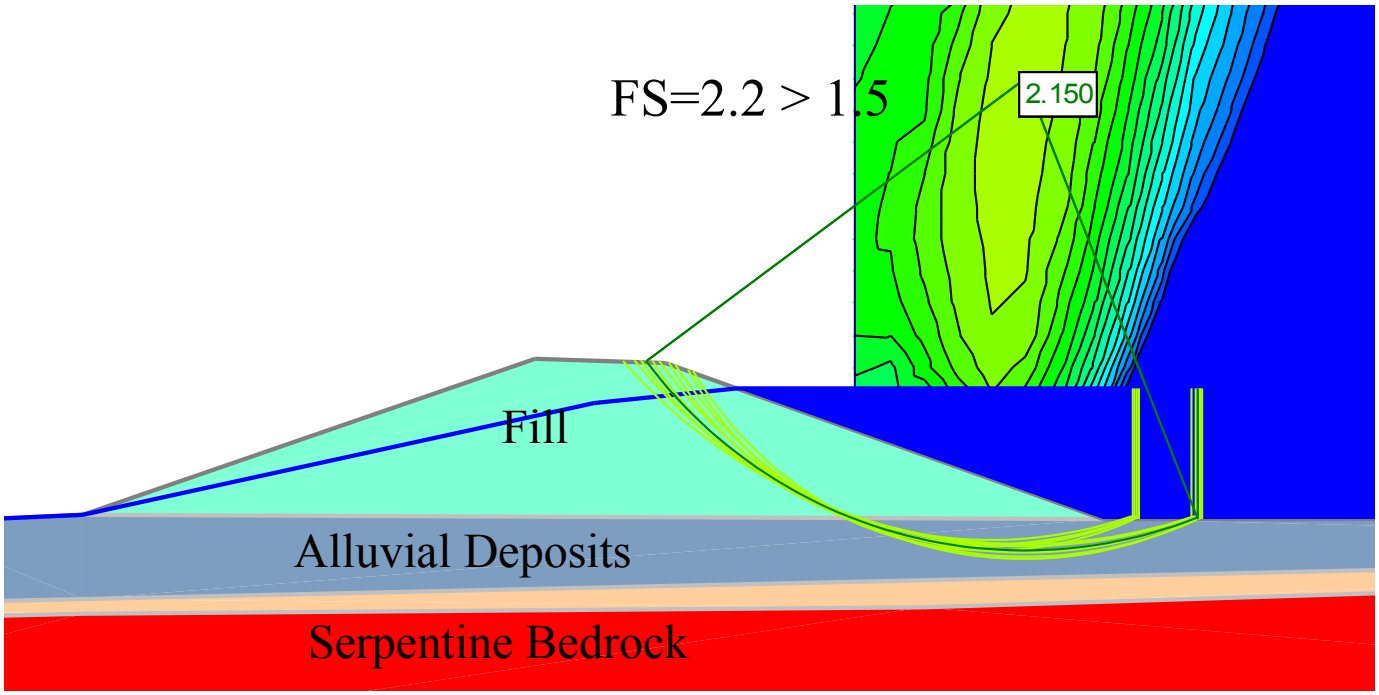
| | | | |
|---|---|---|---|
| COMPILED BY: A. Johns | **CROSS-SECTION B-B** | M2 CONSULTANTS INC | **PLATE** |
| REVISED BY: A. Johns | | | |
| CHECKED BY: Y. Prashar | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | 638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | **22** |
| DATE: MAR 11, 2012 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE: PLATE 7, CROSS-SECTION F | | |

Library file: L:\2012\PLATE 7, CROSS-SECTION F.ppt

PLOTTED: March 21, 2012, 08:49:07 PM




# LONG TERM STATIC CASE WITH RESERVOIR FILLED




| | | | |
|---|---|---|---|
| COMPILED BY: A. Johns | **CROSS-SECTION B-B** | M2 CONSULTANTS inc | **PLATE** |
| REVISED BY: A. Johns | | | |
| CHECKED BY: Y. Prashar | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | 638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | **23** |
| DATE: MAR 11, 2012 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE: PLATE 7, CROSS-SECTION F | | |

Library file: L:\2012\PLATE 7, CROSS-SECTION F.ppt

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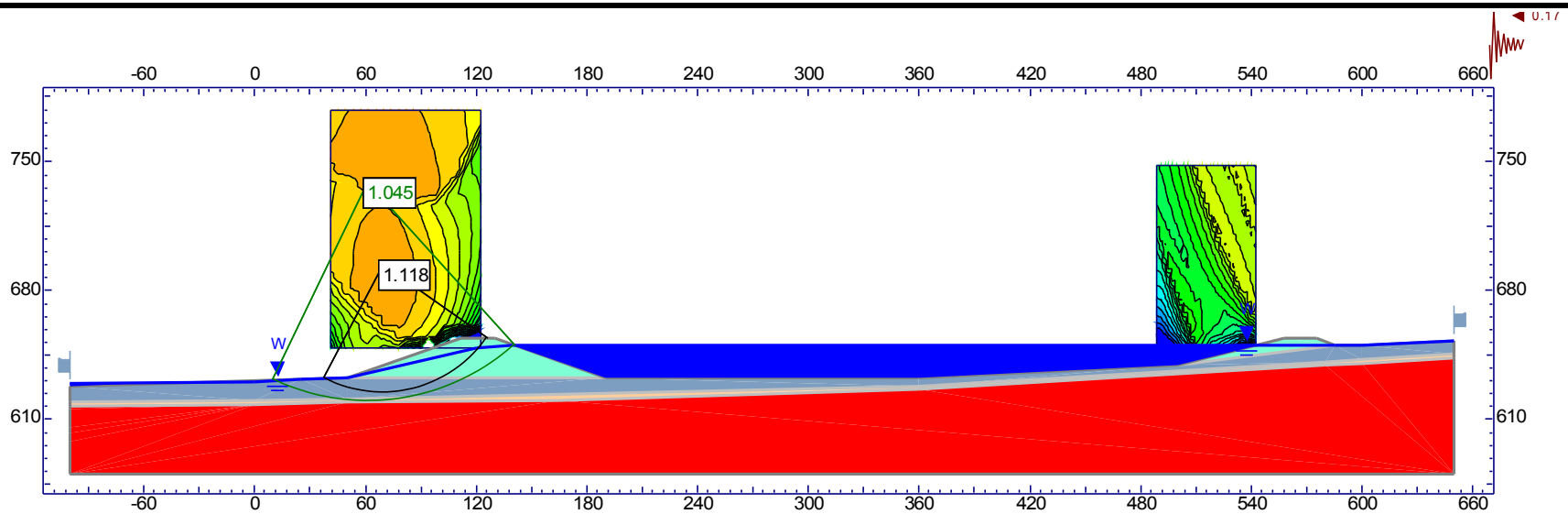


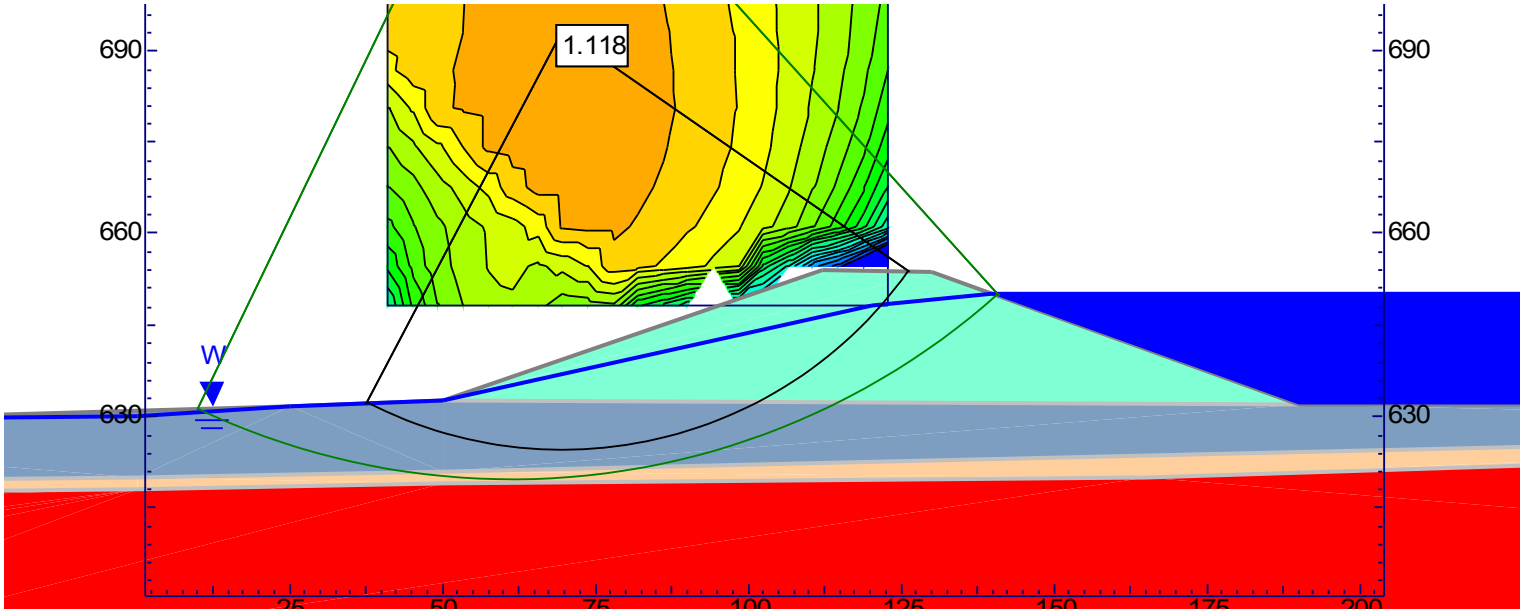


# SHORT TERM PSEUDO DYNAMIC (Kh=0.17): FS = 1.0

| | | | |
|---|---|---|---|
| COMPILED BY: A. Johns | **CROSS-SECTION B-B** | M2 CONSULTANTS inc | **PLATE** |
| REVISED BY: A. Johns | | | |
| CHECKED BY: Y. Prashar | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | 638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | **24** |
| DATE: MAR 11, 2012 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE: PLATE 7, CROSS-SECTION F | | |

Library file: L:\2012\PLATE 7, CROSS-SECTION F.ppt

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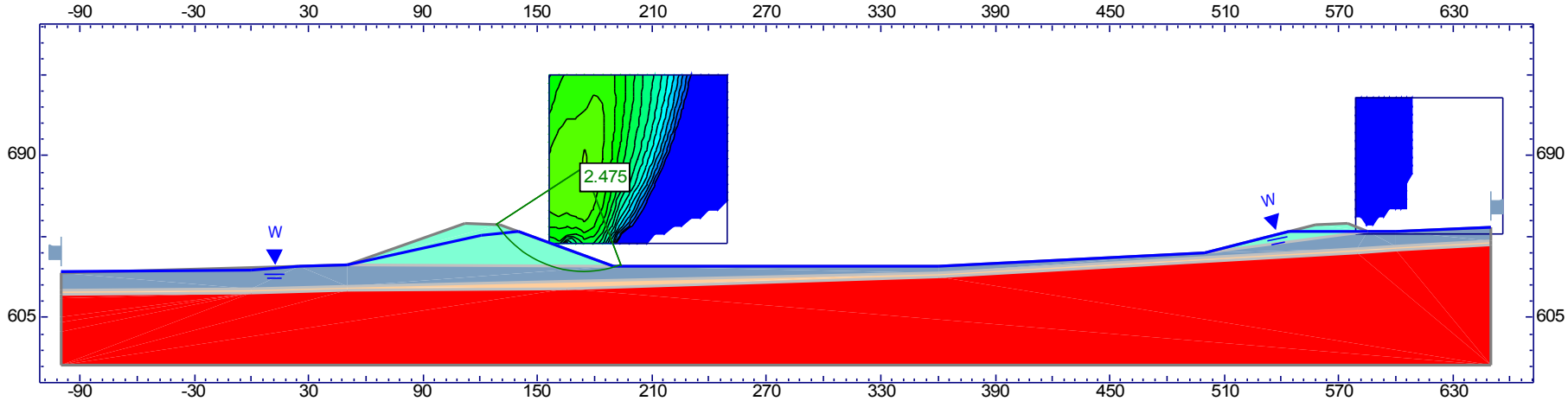


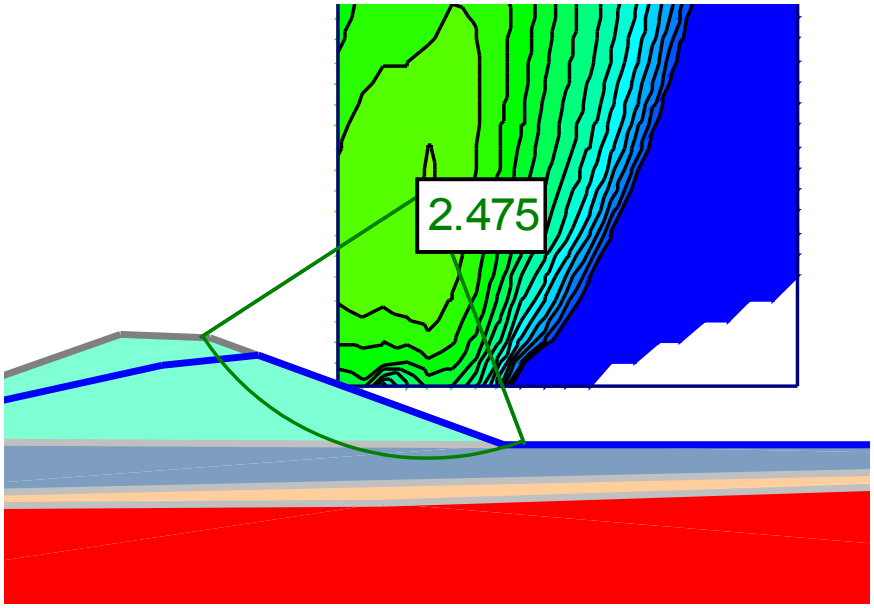


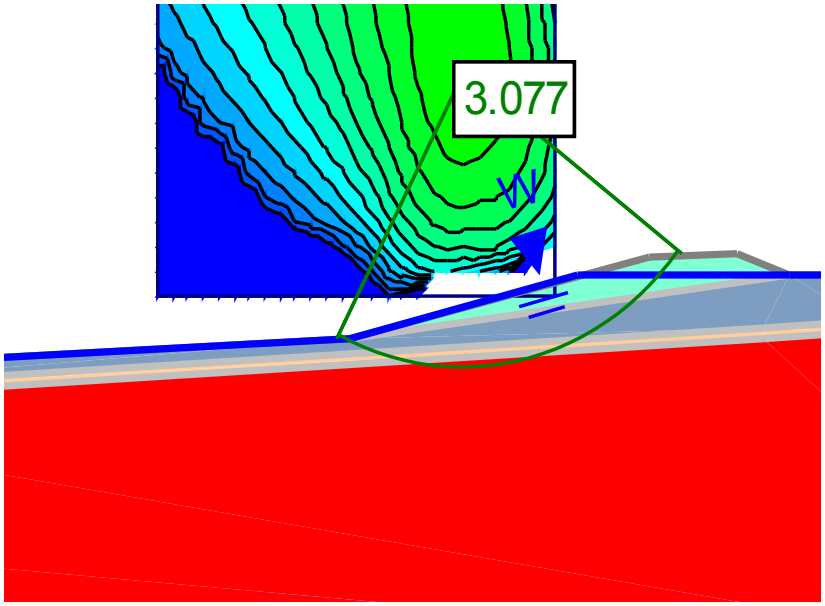


# SHORT TERM RAPID DRAWDOWN CASE

| | | | |
|---|---|---|---|
| COMPILED BY: A. Johns | **CROSS-SECTION B-B** | M2 CONSULTANTS inc<br>638 19th Street, Unit 18<br>San Francisco, California 94107<br>Ph. (415) 244-0761 | **PLATE** |
| REVISED BY: A. Johns | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | | **25** |
| CHECKED BY: Y. Prashar | | | |
| DATE: MAR 11, 2012 / APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO / FILE: PLATE 7, CROSS-SECTION F | | |

Library file: L:\2012\PLATE 7, CROSS-SECTION F.ppt

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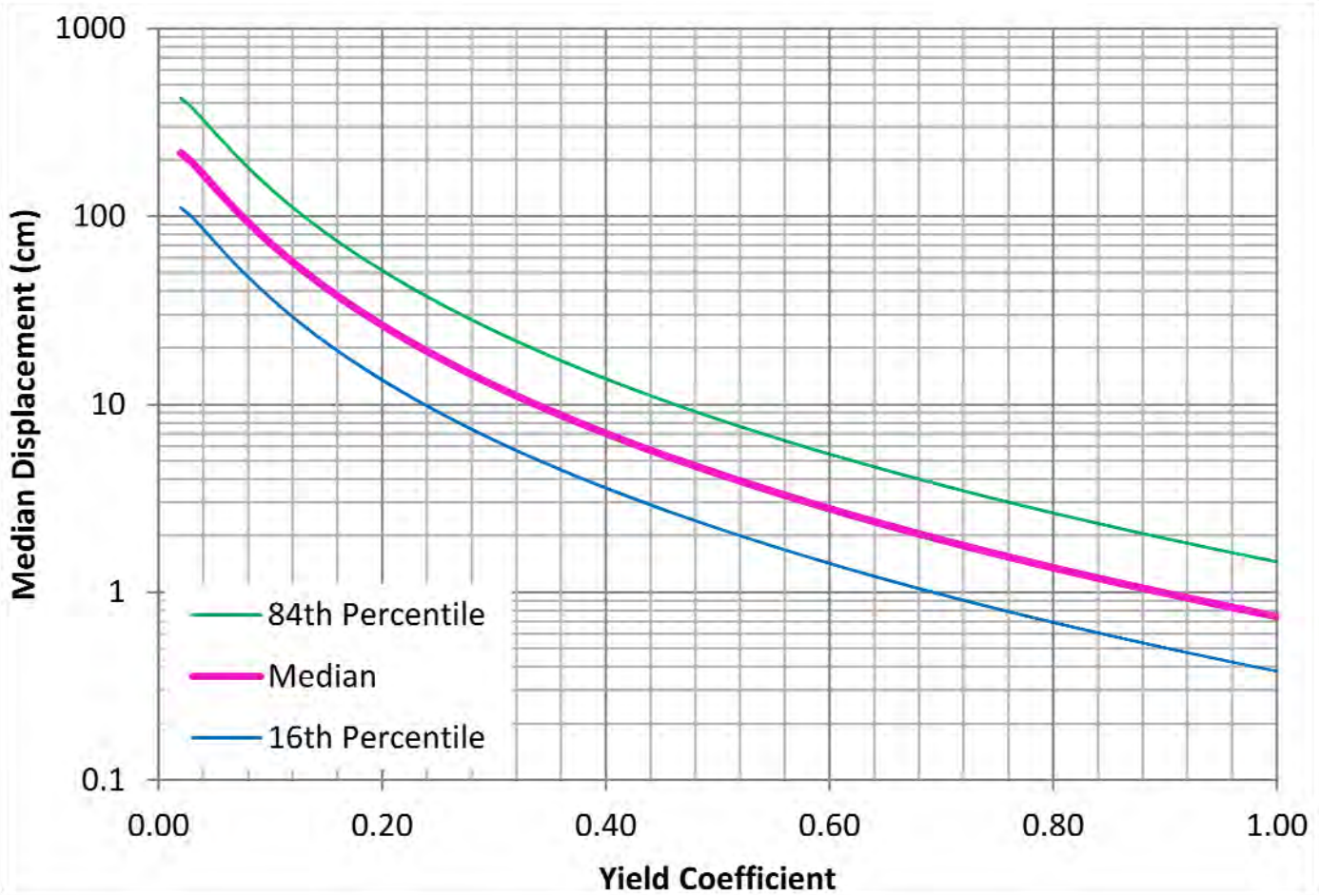
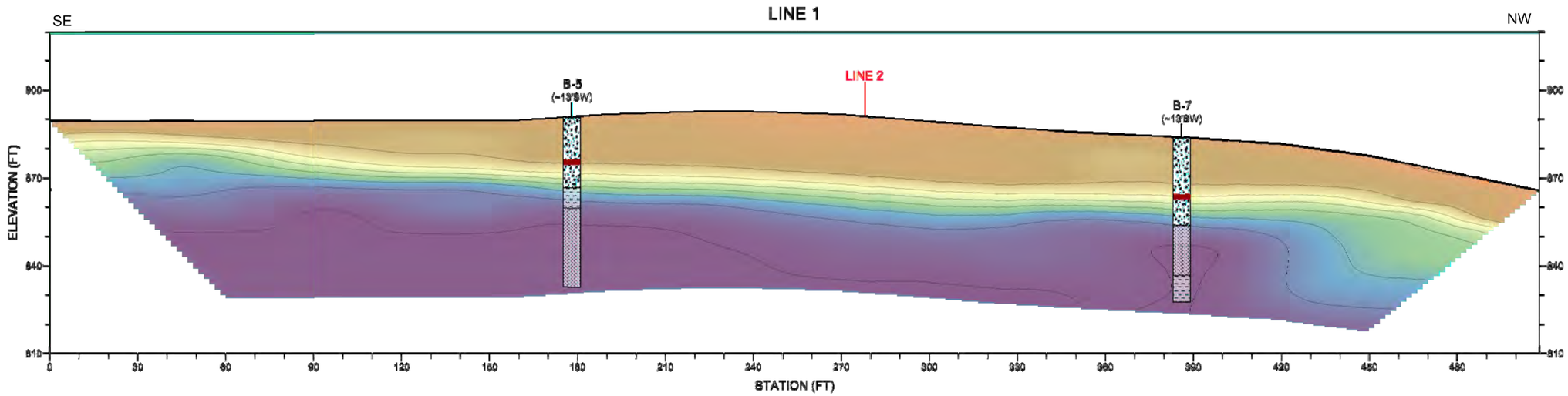
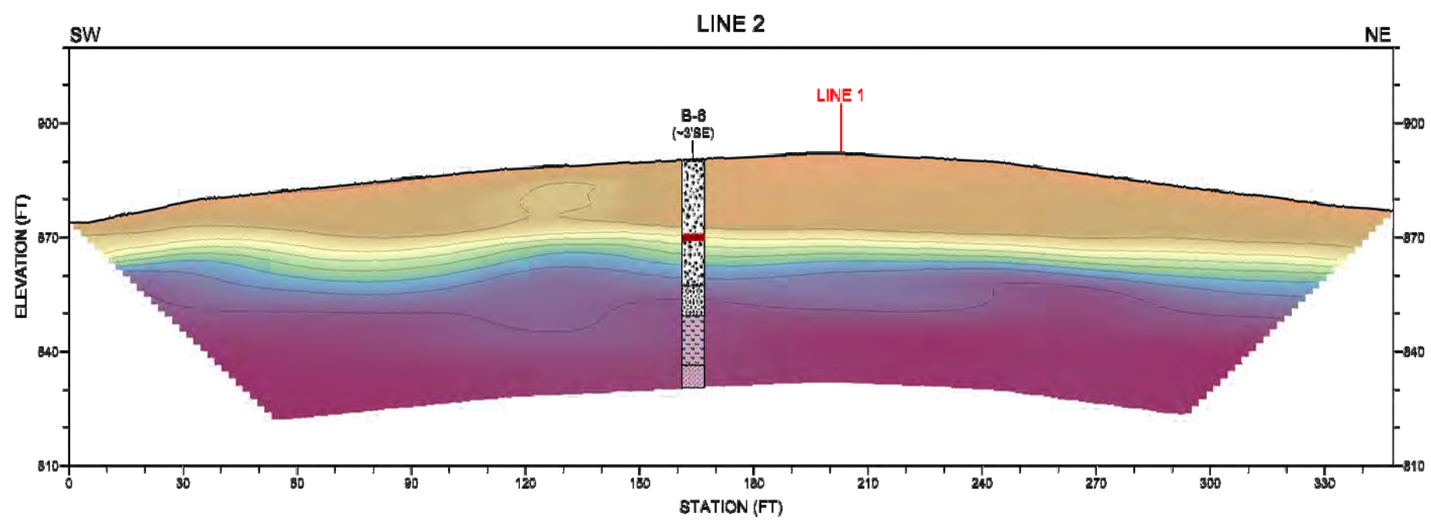
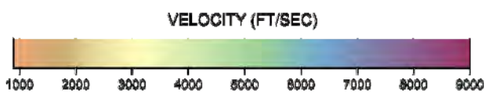



SIMPLIFIED PROCEDURE FOR ESTIMATING EARTHQUAKE INDUCED DEVIATORIC SLOPE DISPLACEMENTS
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geoenvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

| | | | | |
|---|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | | SITE 3A EARTHQUAKE INDUCED SLOPE DISPLACEMENTS | M2 CONSULTANTS inc<br>638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | | 26 |
| CHECKED BY: A. JOHNS | | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/24/2013 | APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 | | |




| BORING LOGS | |
|---|---|
| | CONGLOMERATE |
| | GRAVEL |
| | SHALE |
| | GRAYWACKE |
| | SIGNIFICANT INCREASE IN SAMPLING BLOW COUNTS |

SCALE IN FEET

0 10 20 40 80

| | |
|---|---|
| COMPILED BY: | N. SCHEEMAKER |
| REVISED BY: | N. SCHEEMAKER |
| CHECKED BY: | A. JOHNS |
| DATE: 01/19/2013 | APPROVED BY: Y. PRASHAR |

# SITE 3C-2 SHEAR WAVE VELOCITY PROFILE

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / GEO 2

SCALE: 1 inch:40 feet

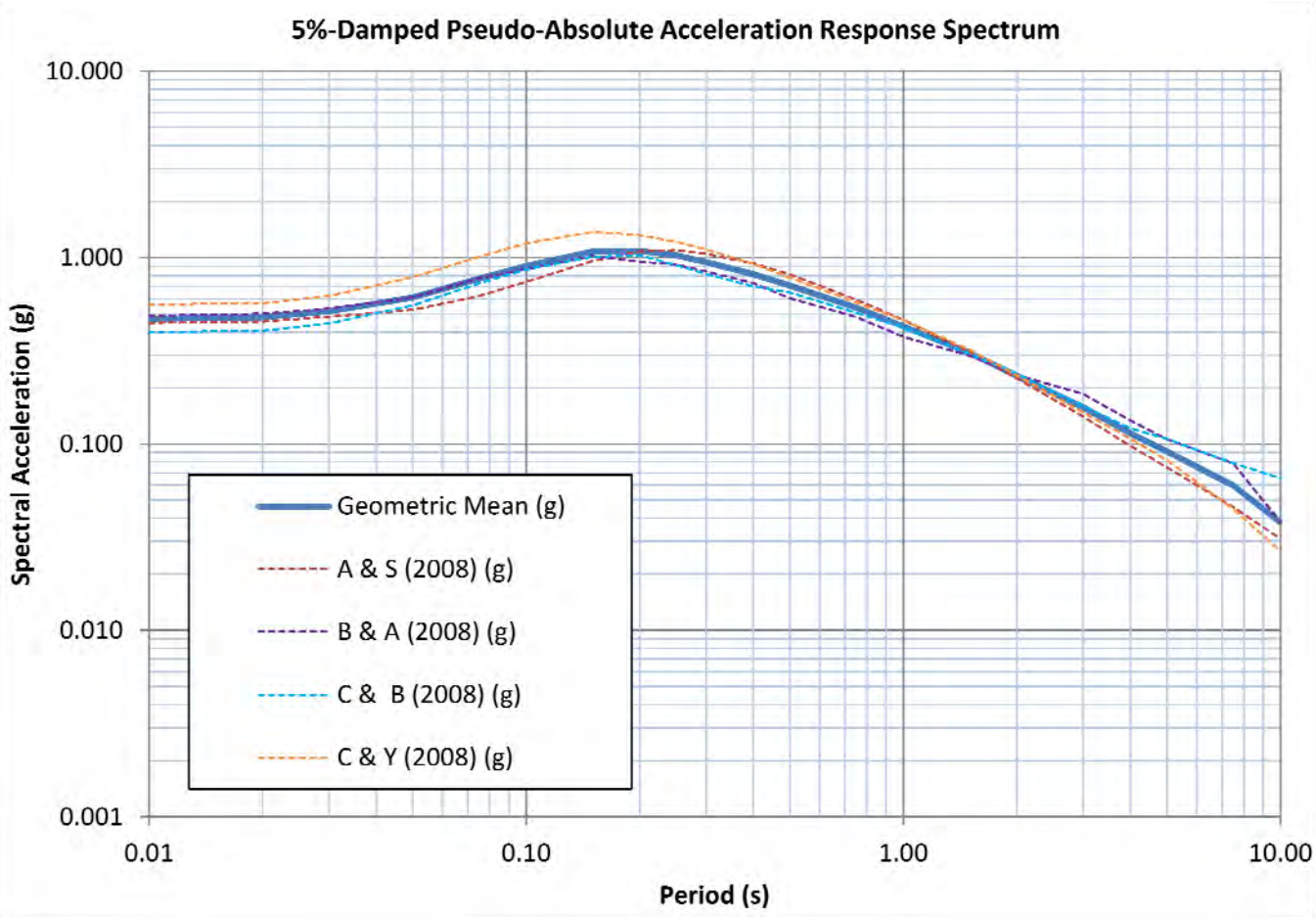
M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

PLATE NO.
27

| PEER Ground Motion Database Target Spectrum Report 21-Jan-2013 | | | | | |
|---|---|---|---|---|---|
| Earthquake Source Parameters | | | | Distance Parameters | |
| Mag | 7.9 | Strike Slip Faulting | | Rrup (km) | 12.95 |
| Dip (deg) | 90 | $V_{S30}$ (m/s) | 760 | Rjb (km) | 12.95 |
| Width (km) | 30 | | | Rx (km) | 12.95 |

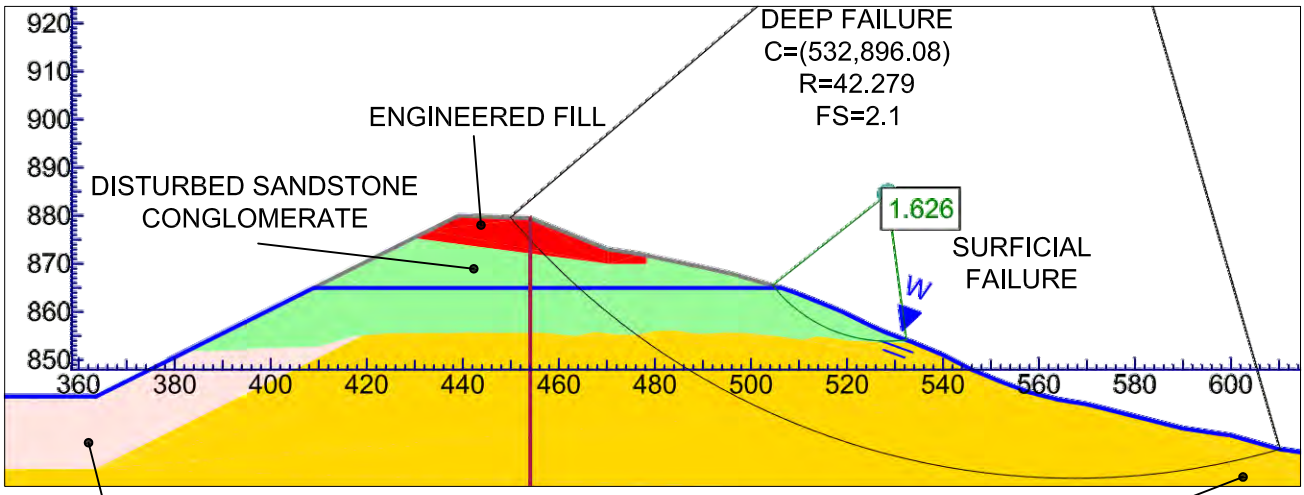
| Period (s) | Geometric Mean (g) | A & S (2008) (g) | B & A (2008) (g) | C & B (2008) (g) | C & Y (2008) (g) |
|---|---|---|---|---|---|
| 0.01 | 0.469 | 0.446 | 0.487 | 0.397 | 0.560 |
| 0.02 | 0.479 | 0.454 | 0.499 | 0.406 | 0.570 |
| 0.03 | 0.515 | 0.482 | 0.531 | 0.443 | 0.621 |
| 0.05 | 0.611 | 0.524 | 0.612 | 0.554 | 0.784 |
| 0.08 | 0.771 | 0.621 | 0.776 | 0.726 | 1.010 |
| 0.10 | 0.902 | 0.740 | 0.866 | 0.864 | 1.193 |
| 0.15 | 1.079 | 0.960 | 1.021 | 1.007 | 1.374 |
| 0.20 | 1.086 | 1.079 | 0.950 | 1.030 | 1.319 |
| 0.25 | 1.026 | 1.098 | 0.915 | 0.908 | 1.213 |
| 0.30 | 0.945 | 1.047 | 0.842 | 0.816 | 1.106 |
| 0.40 | 0.815 | 0.933 | 0.729 | 0.700 | 0.924 |
| 0.50 | 0.708 | 0.814 | 0.606 | 0.650 | 0.782 |
| 0.75 | 0.537 | 0.595 | 0.481 | 0.506 | 0.573 |
| 1.00 | 0.429 | 0.463 | 0.377 | 0.421 | 0.461 |
| 1.50 | 0.308 | 0.310 | 0.297 | 0.304 | 0.320 |
| 2.00 | 0.233 | 0.225 | 0.235 | 0.238 | 0.233 |
| 3.00 | 0.157 | 0.140 | 0.185 | 0.160 | 0.147 |
| 4.00 | 0.114 | 0.098 | 0.134 | 0.122 | 0.107 |
| 5.00 | 0.091 | 0.075 | 0.106 | 0.107 | 0.082 |
| 7.50 | 0.060 | 0.046 | 0.079 | 0.079 | 0.045 |
| 10.00 | 0.038 | 0.031 | 0.038 | 0.065 | 0.027 |

| PEER Ground Motion Database Target Spectrum Report 21-Jan-2013 | | | | | |
|---|---|---|---|---|---|
| Earthquake Source Parameters | | | | Distance Parameters | |
| Mag | 7.9 | Strike Slip Faulting | | Rrup (km) | 12.95 |
| Dip (deg) | 90 | $V_{S30}$ (m/s) | 760 | Rjb (km) | 12.95 |
| Width (km) | 30 | | | Rx (km) | 12.95 |




| CALCULATION OF WEIGHTED AVERAGE 2008 NGA MODELS: | | | | | |
|---|---|---|---|---|---|
| NGA Model: | AS08 | BA08 | CB08 | CY08 | I08 |
| Weight: | 0.25 | 0.25 | 0.25 | 0.25 | 0 |

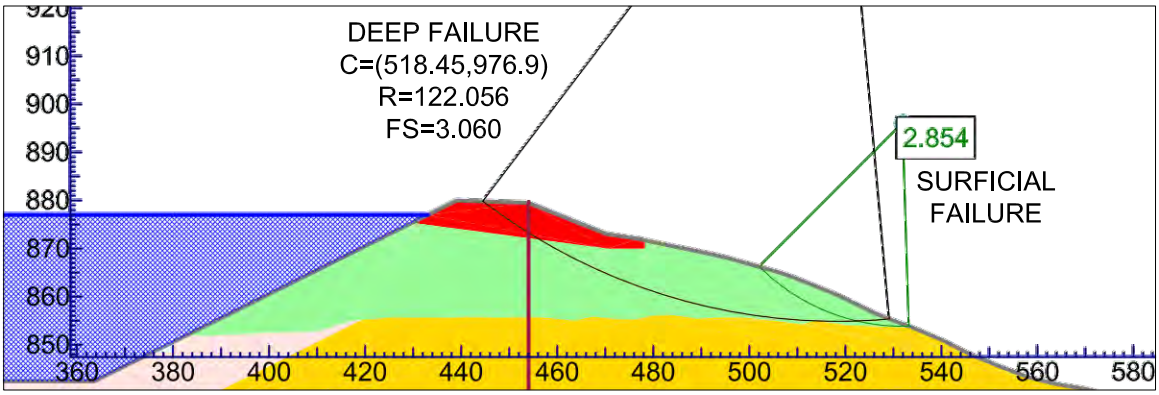
| | | | | |
|---|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | | 3C-2 SITE SPECIFIC RESPONSE SPECTRA<br>2008 NGA ATTENUATION EQUATIONS | M2 CONSULTANTS inc<br>638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | PLATE NO.<br>29 |
| REVISED BY: N. SCHEEMAKER | | | | |
| CHECKED BY: A. JOHNS | | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/21/2013 | APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 | | |



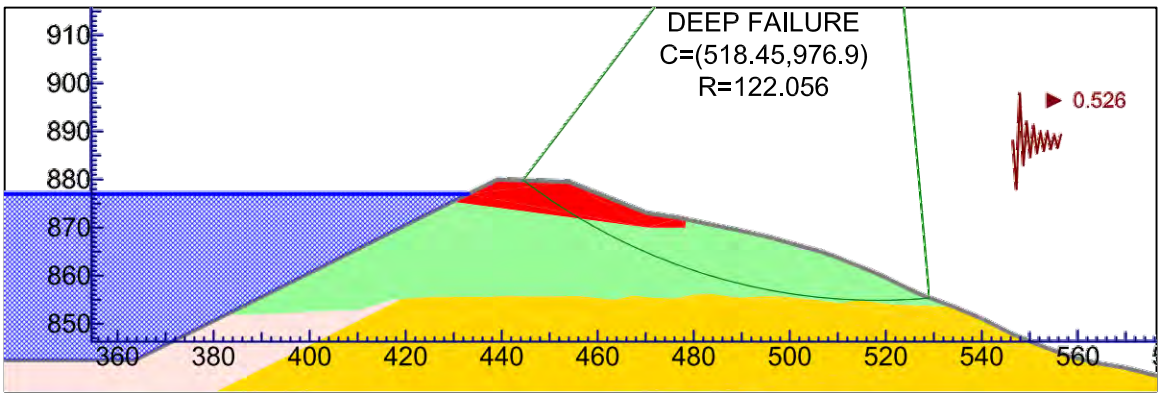

DISTURBED LAYERS OF GRAYWACKE AND SHALE

CONSTRUCTION CONDITIONS - RESERVOIR EMPTY, GROUND WATER AT ELEVATION 865 FEET
F.S.=2.100

LAYERS OF GRAYWACKE AND SHALE

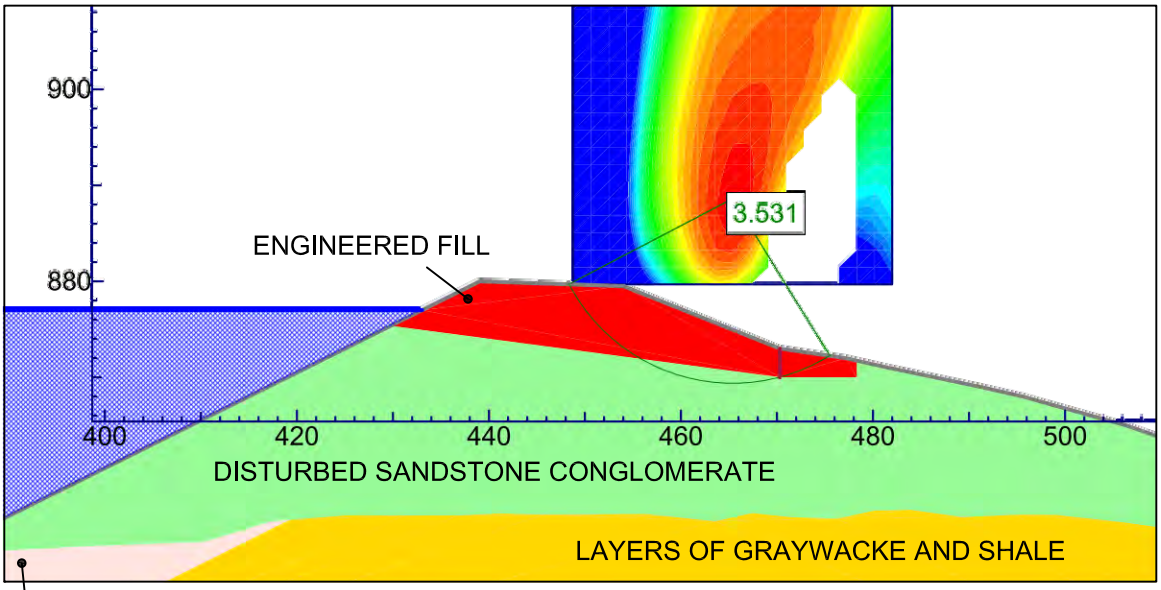



LONG TERM CONDITIONS - RESERVOIR FILLED, NO LEAKAGE FROM RESERVOIR
F.S.=3.060



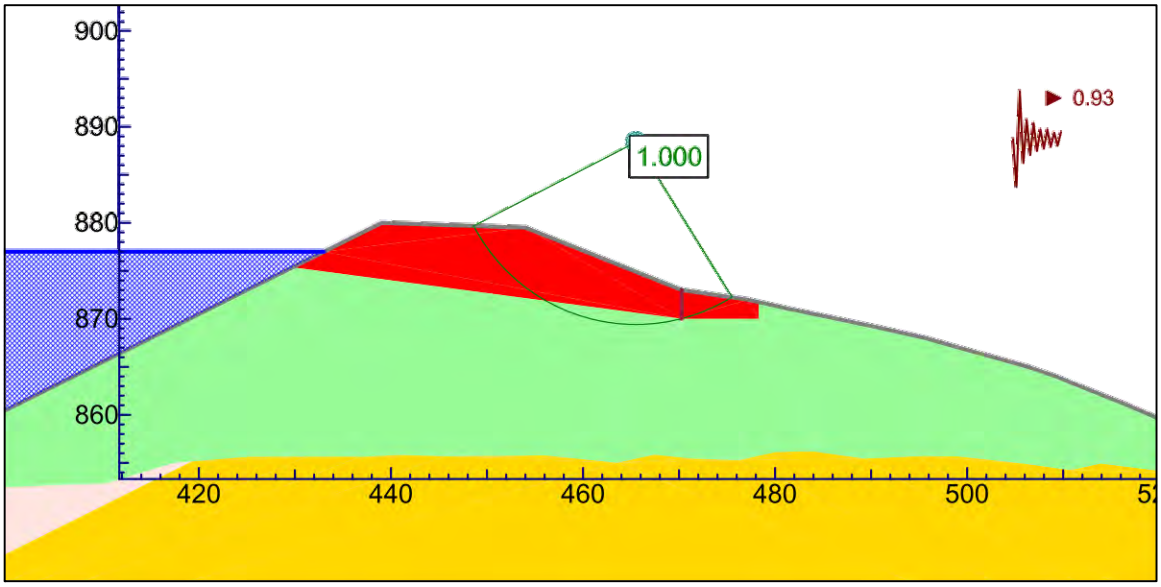

PSEUDO DYNAMIC - RESERVOIR FILLED
F.S.=1.0, Kh=0.526

| | | | |
|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | **3C-2 STABILITY ANALYSIS NORTHERN EMBANKMENT DOWNSTREAM (DEEP)** | M2 CONSULTANTS inc | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | 30 |
| CHECKED BY: A. JOHNS | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | | |
| DATE: 01/21/2013 / APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 / SCALE 1inch:40feet | 638 19th STREET, UNIT 18 SAN FRANCISCO, CALIFORNIA, 94107 | |



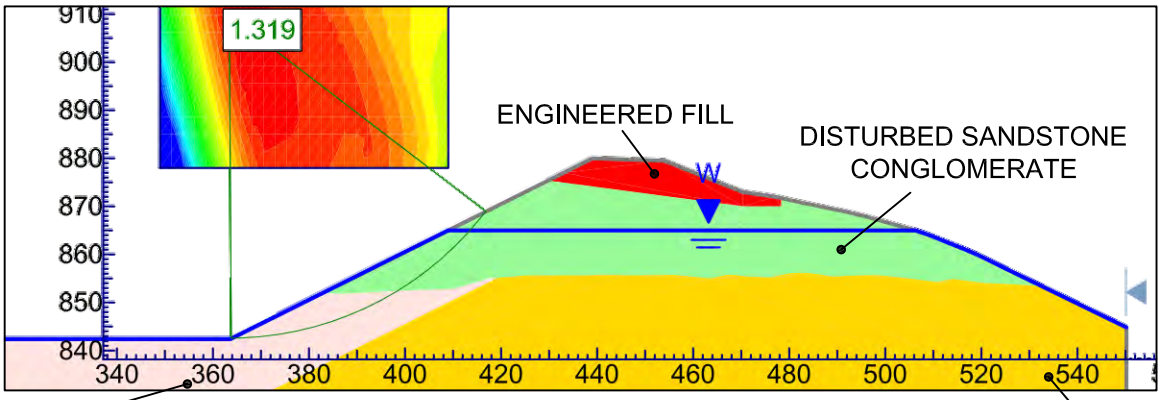

DISTURBED LAYERS OF GRAYWACKE AND SHALE

LONG TERM CONDITIONS - RESERVOIR FILLED, NO LEAKAGE FROM RESERVOIR
F.S.=3.531




PSEUDO DYNAMIC - RESERVOIR FILLED
F.S.=1.0, Kh=0.93

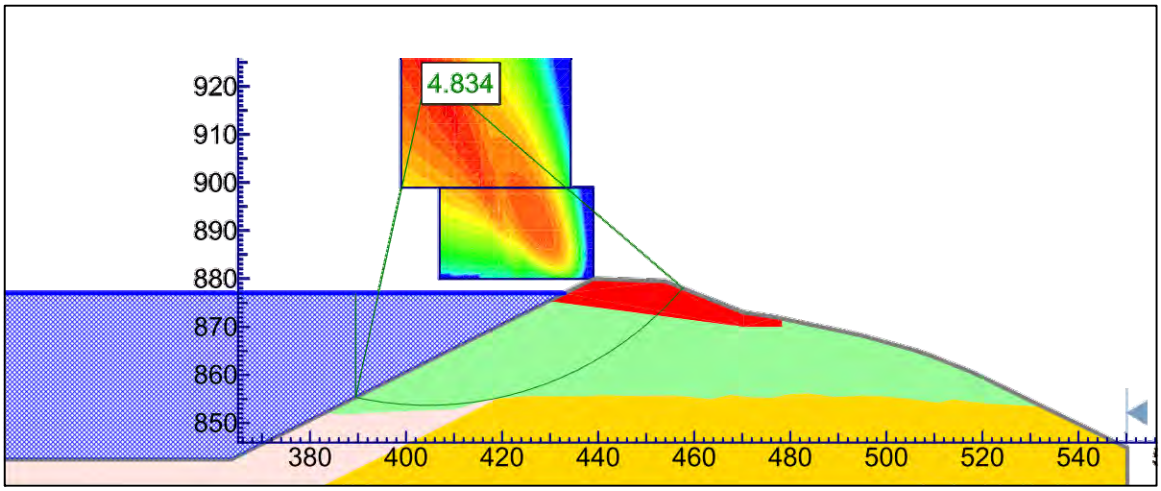
| | | | | |
|---|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | | 3C-2 STABILITY ANALYSIS<br>NORTHERN EMBANKMENT DOWNSTREAM (FILL) | M2 CONSULTANTS inc<br>638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | | 31 |
| CHECKED BY: A. JOHNS | | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/21/2013 | APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 | SCALE 1inch:20feet | |



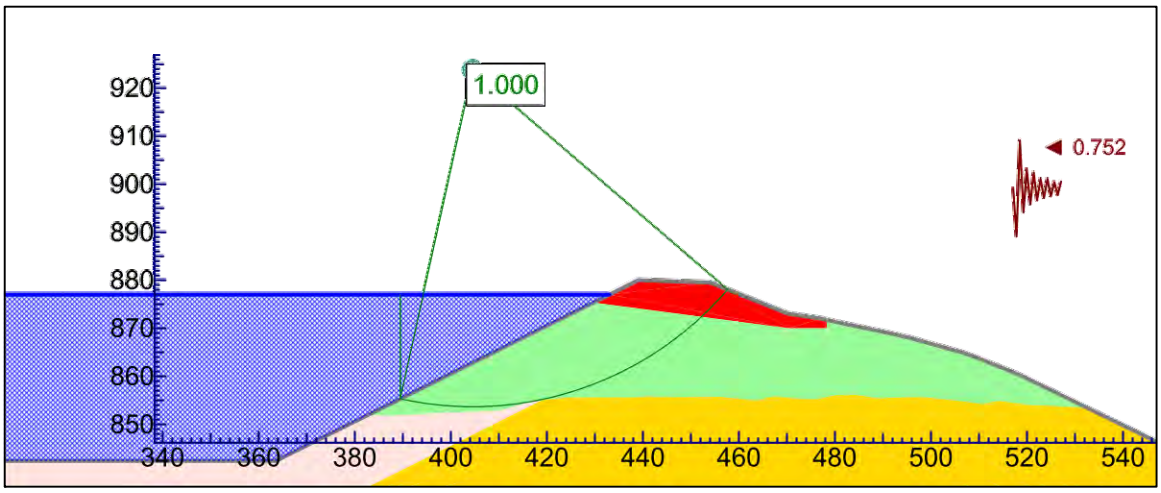

DISTURBED LAYERS OF GRAYWACKE AND SHALE

CONSTRUCTION CONDITIONS - RESERVOIR EMPTY, GROUND WATER AT ELEVATION 865 FEET
F.S.=1.319

LAYERS OF GRAYWACKE AND SHALE

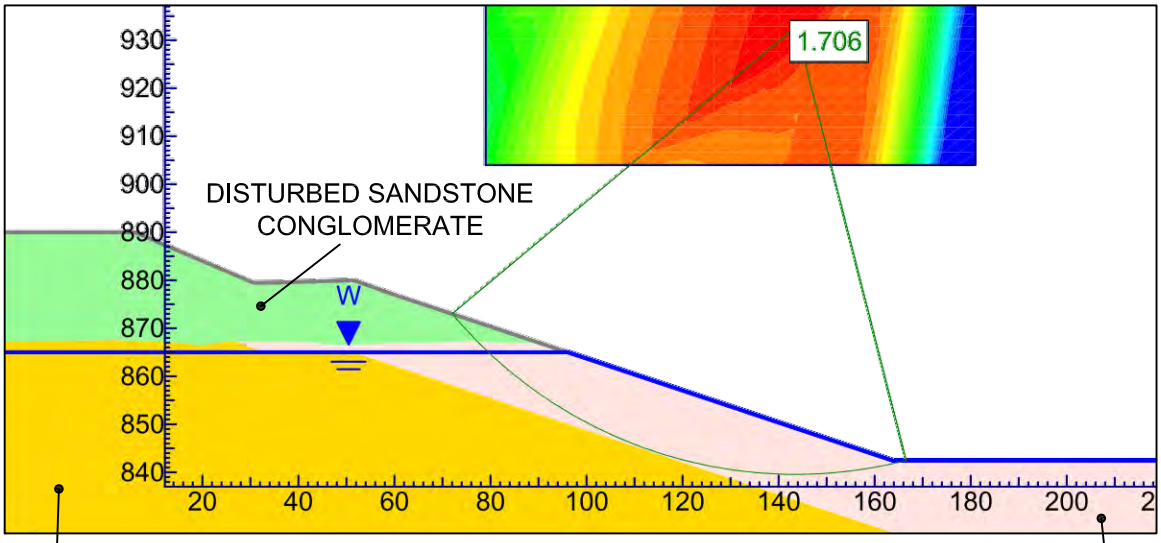



LONG TERM CONDITIONS - RESERVOIR FILLED, NO LEAKAGE FROM RESERVOIR
F.S.=4.834




PSEUDO DYNAMIC - RESERVOIR FILLED
F.S.=1.0, Kh=0.752

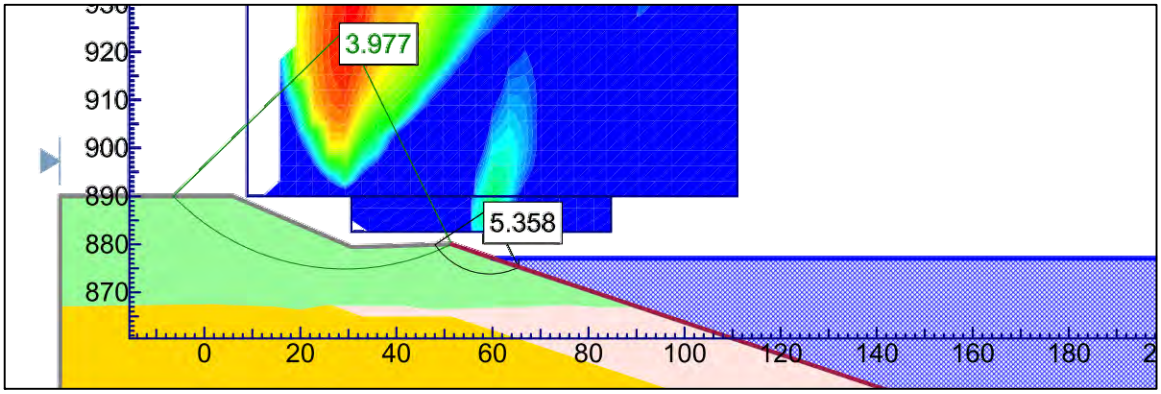
| | | | | |
|---|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | | 3C-2 STABILITY ANALYSIS<br>NORTHERN EMBANKMENT UPSTREAM | M2 CONSULTANTS inc<br>638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | PLATE NO.<br>32 |
| REVISED BY: N. SCHEEMAKER | | | | |
| CHECKED BY: A. JOHNS | | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/21/2013 | APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 | SCALE 1inch:40feet | |



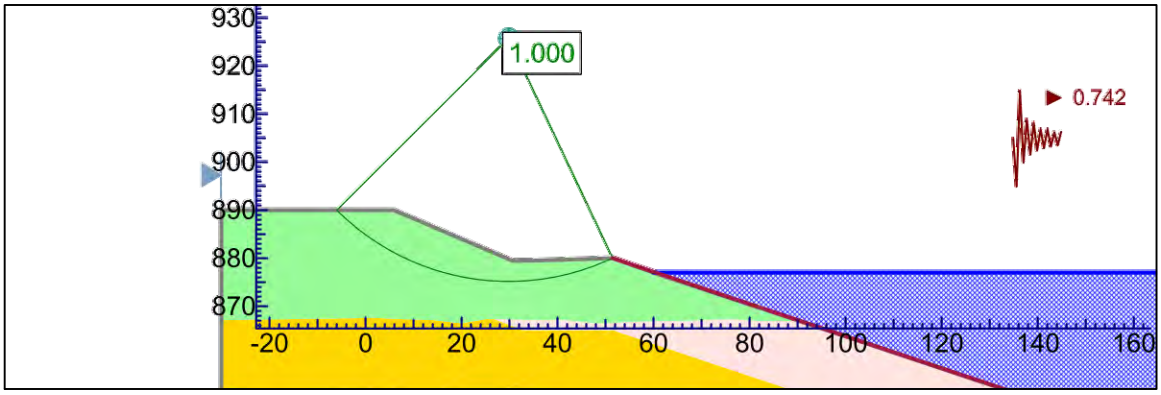

LAYERS OF GRAYWACKE AND SHALE

CONSTRUCTION CONDITIONS - RESERVOIR EMPTY, GROUND WATER AT ELEVATION 865 FEET
F.S.=1.706

DISTURBED LAYERS OF GRAYWACKE AND SHALE

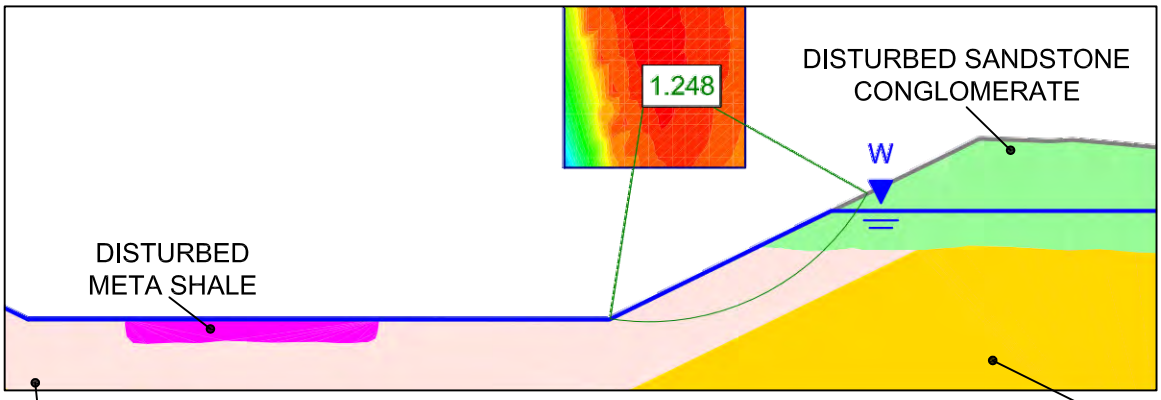



LONG TERM CONDITIONS - RESERVOIR FILLED, NO LEAKAGE FROM RESERVOIR
F.S.=3.977

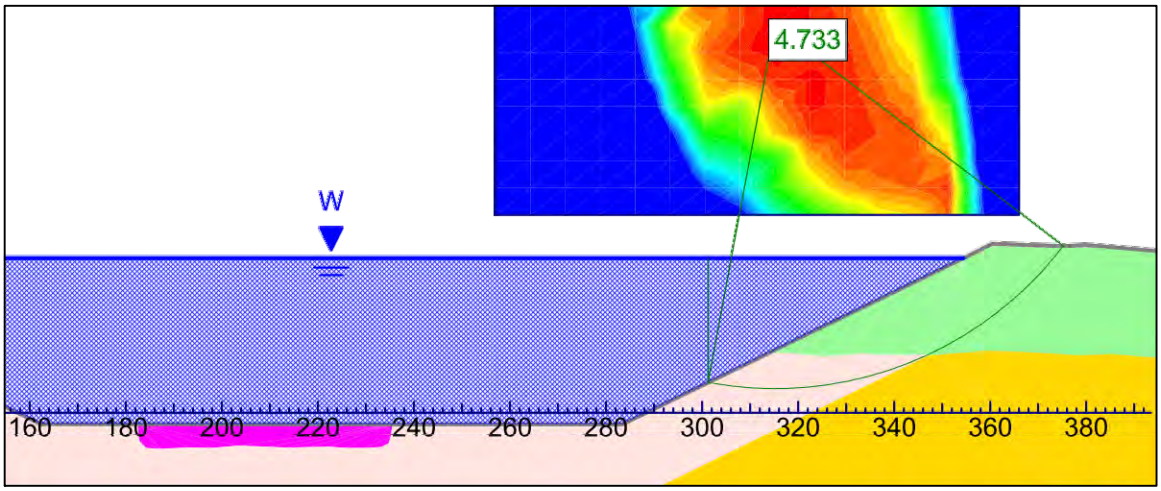



PSEUDO DYNAMIC - RESERVOIR FILLED
F.S.=1.0, Kh=0.742

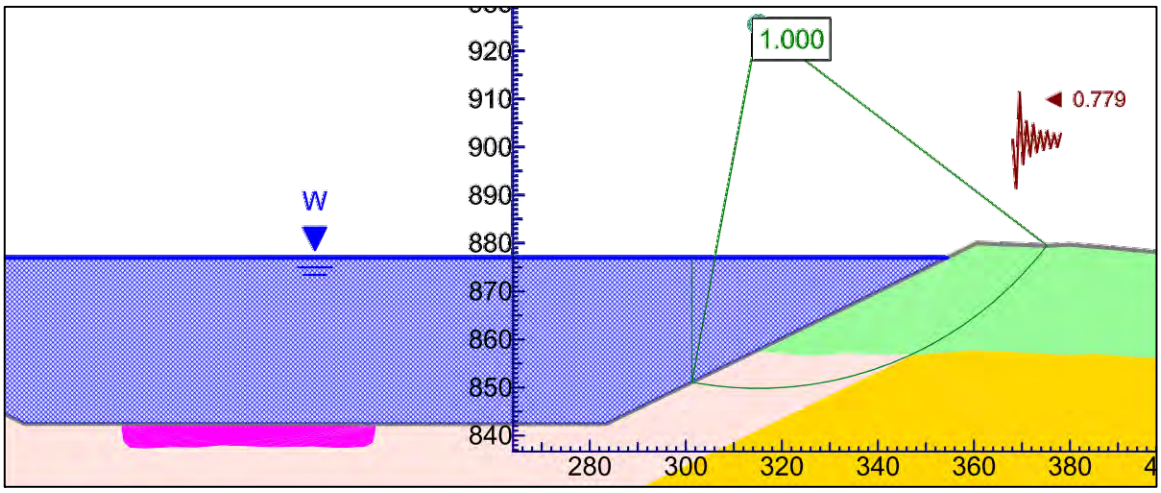
| | | | | |
|---|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | | **3C-2 STABILITY ANALYSIS SOUTHERN EMBANKMENT UPSTREAM** | M2 CONSULTANTS inc | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | | 33 |
| CHECKED BY: A. JOHNS | | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | | |
| DATE: 01/21/2013 | APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 | SCALE 1inch:40feet | 638 19th STREET, UNIT 18 SAN FRANCISCO, CALIFORNIA, 94107 |




CONSTRUCTION CONDITIONS - RESERVOIR EMPTY, GROUND WATER AT ELEVATION 865 FEET
F.S.=1.248

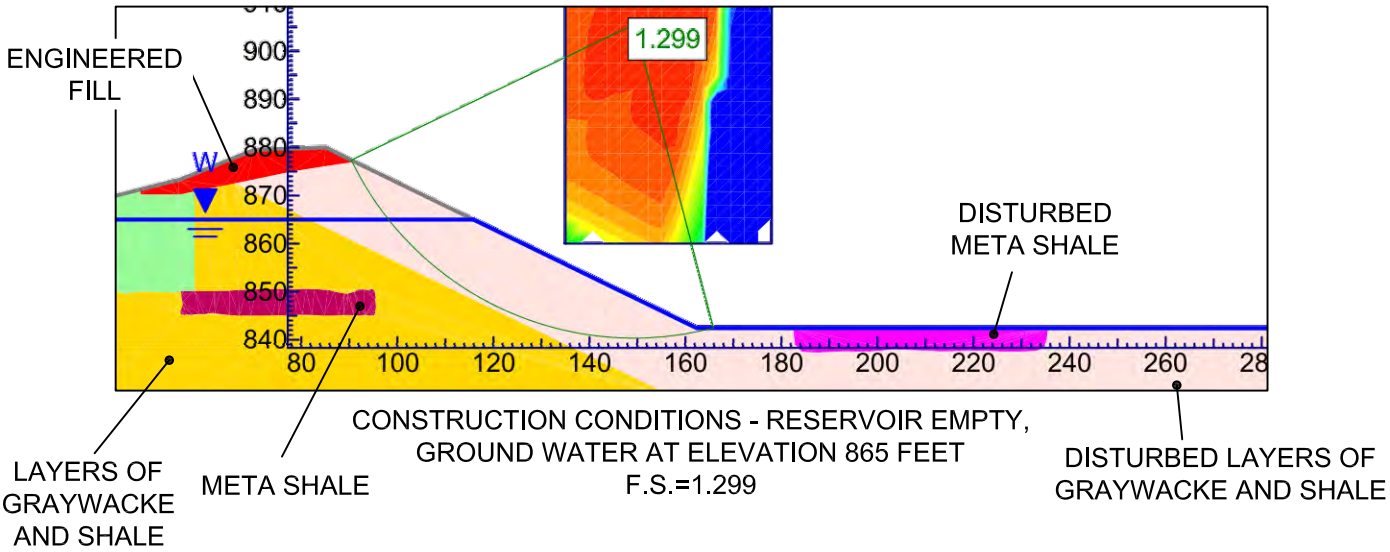



LONG TERM CONDITIONS - RESERVOIR FILLED, NO LEAKAGE FROM RESERVOIR
F.S.=4.733

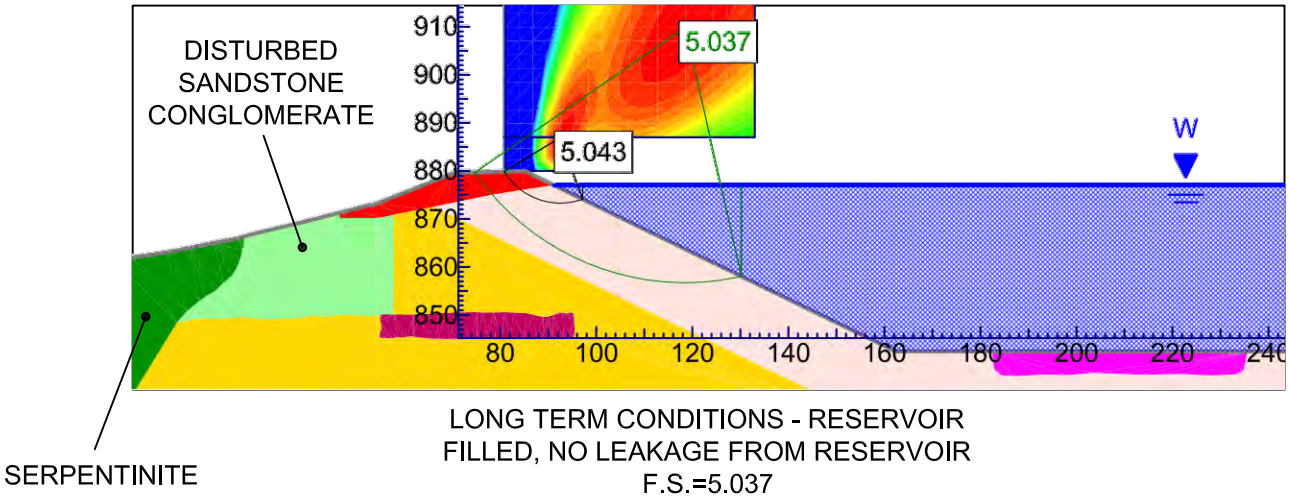



PSEUDO DYNAMIC - RESERVOIR FILLED
F.S.=1.0, Kh=0.779

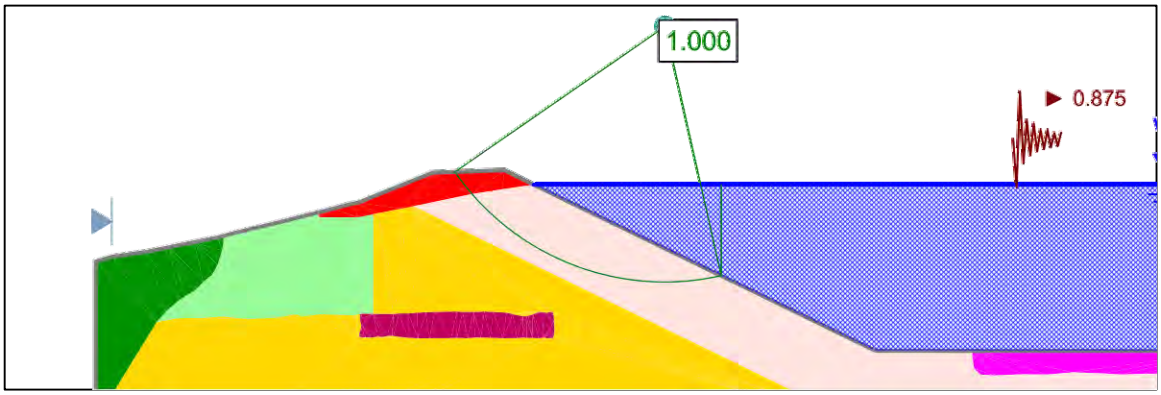
| | | | |
|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | **3C-2 STABILITY ANALYSIS EASTERN EMBANKMENT UPSTREAM** | M2 CONSULTANTS inc | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | 34 |
| CHECKED BY: A. JOHNS | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | | |
| DATE: 01/21/2013 / APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 / SCALE 1inch:40feet | 638 19th STREET, UNIT 18 SAN FRANCISCO, CALIFORNIA, 94107 | |




CONSTRUCTION CONDITIONS - RESERVOIR EMPTY, GROUND WATER AT ELEVATION 865 FEET
F.S.=1.299

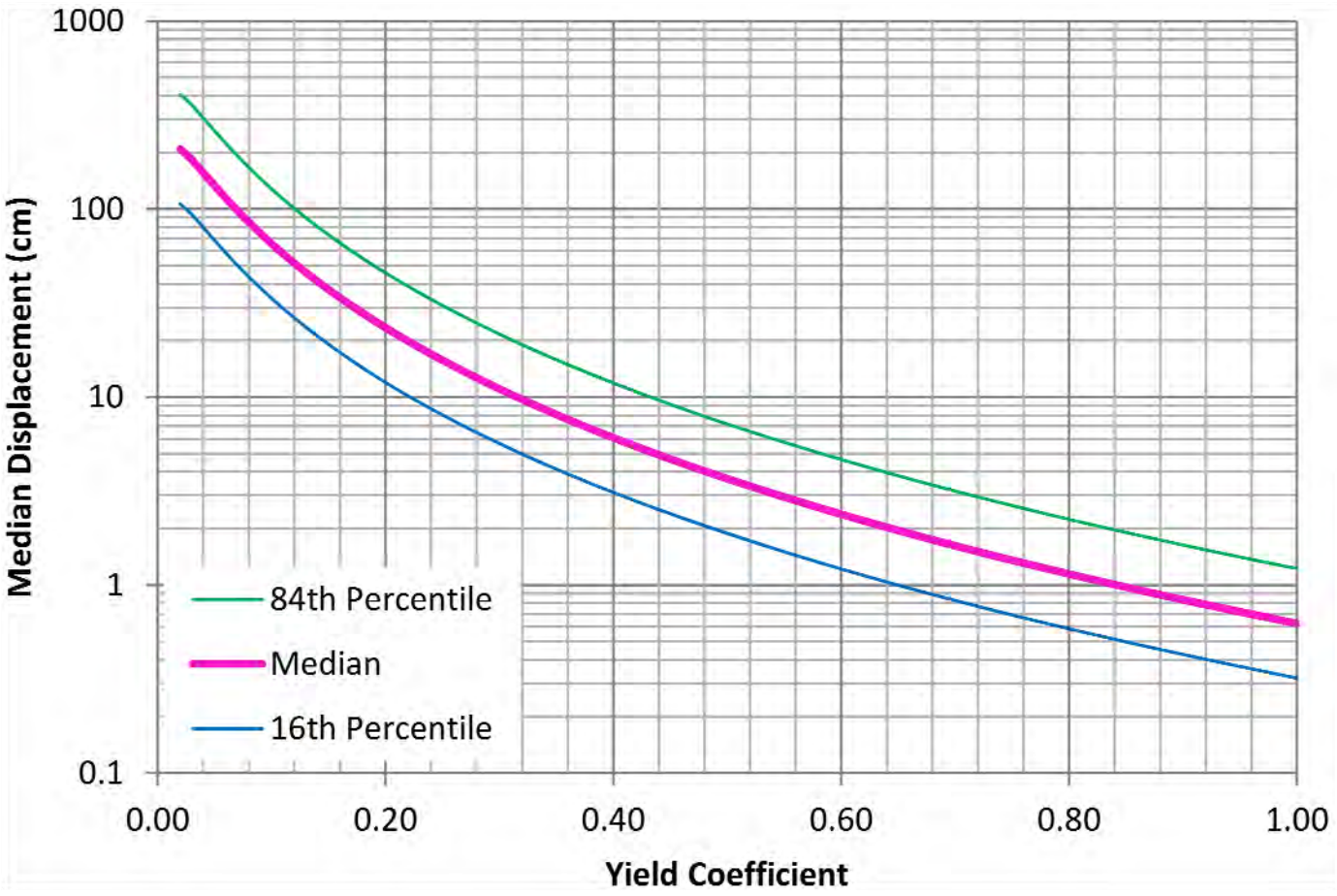



LONG TERM CONDITIONS - RESERVOIR FILLED, NO LEAKAGE FROM RESERVOIR
F.S.=5.037




PSEUDO DYNAMIC - RESERVOIR FILLED
F.S.=1.0, Kh=0.875

| | | | |
|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | **3C-2 STABILITY ANALYSIS WESTERN EMBANKMENT UPSTREAM** | M2 CONSULTANTS inc | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | 35 |
| CHECKED BY: A. JOHNS | **OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA** | | |
| DATE: 01/21/2013 / APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 / SCALE 1inch:40feet | 638 19th STREET, UNIT 18 SAN FRANCISCO, CALIFORNIA, 94107 | |




SIMPLIFIED PROCEDURE FOR ESTIMATING EARTHQUAKE INDUCED DEVIATORIC SLOPE DISPLACEMENTS
by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geoenvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

| | | | | |
|---|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | | SITE 3C-2 EARTHQUAKE INDUCED SLOPE DISPLACEMENTS | M2 CONSULTANTS inc<br>638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | | 36 |
| CHECKED BY: A. JOHNS | | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/24/2013 | APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 | | |

# APPENDIX A

# SITE 3A

# BORING LEGEND &

# BORING LOGS B-1 THROUGH B-4

| MAJOR DIVISIONS | | | SYMBOLS | SECONDARY DIVISIONS |
|---|---|---|---|---|
| COARSE GRAINED SOILS<br>More than half is larger than No. 28 sieve | GRAVELS<br>More than half of coarse fraction retained on No. 4 sieve | CLEAN GRAVELS (Little or no fines) | GW | Well-graded gravels, gravel-sand mixtures, little or no fines. |
| | | | GP | Poorly-graded gravels, gravel-sand mixtures, little or no fines. |
| | | GRAVELS WITH FINES (Appreciable amount of fines) | GM | Silty gravels, gravel-sand-silt mixtures. |
| | | | GC | Clayey gravels, gravel-sand-clay mixtures. |
| | SANDS<br>More than half of coarse fraction passing on No. 4 sieve | CLEAN SANDS (Little or no fines) | SW | Well-graded sands, gravelly sands, little or no fines. |
| | | | SP | Poorly-graded sands, gravelly sands, little or no fines. |
| | | SANDS WITH FINES (Appreciable amount of fines) | SM | Silty sands, sand-silt mixtures. |
| | | | SC | Clayey sands, sand-clay mixtures. |
| FINE GRAINED SOILS<br>More than half is smaller than No. 200 sieve | SILTS AND CLAYS | Liquid Limit less than 50 | ML | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity. |
| | | | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. |
| | | | OL | Organic silts and organic silty clays of low plasticity. |
| | SILTS AND CLAYS | Liquid Limit greater than 50 | MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts. |
| | | | CH | Inorganic clays of high plasticity, fat clays. |
| | | | OH | Organic clays of medium to high plasticity, organic silts. |
| HIGHLY ORGANIC SOILS | | | PT | Peat and other highly organic soils. |

Note: Dual symbols are used to indicate borderline soil classifications

## GRAIN SIZE

U.S. STANDARD SERIES SIEVE

| | #200 | #40 | #10 | #4 | 3/4" | 3" | 12" | |
|---|---|---|---|---|---|---|---|---|
| SILTS AND CLAYS DISTINGUISHED ON BASIS OF PLASTICITY | SAND | | | GRAVEL | | COBBLES | BOULDERS | |
| | FINE | MEDIUM | COARSE | FINE | COARSE | | | |
| | 0.075 mm | 0.475 mm | 2.00 mm | 4.75 mm | 19 mm | 75 mm | 300 mm | |

## PENETRATION RESISTANCE

| SANDS AND GRAVELS | | CLAYS AND SILTS | | |
|---|---|---|---|---|
| IN SITU DENSITY | SPT BLOWS/FOOT* | CONSISTENCY | SPT BLOWS/FOOT* | POCKET PENETROMETER |
| Very loose | 0 - 4 | Very Soft | 0 - 2 | 0 - 500 psf |
| Loose | 4 - 10 | Soft | 2 - 4 | 500 - 1000 psf |
| Medium dense | 10 - 30 | Firm/Medium stiff | 4 - 8 | 1000 - 2000 psf |
| Dense | 30 - 50 | Stiff | 8 - 16 | 2000 - 4000 psf |
| Very dense | Over 50 | Very Stiff | 16 - 32 | 4000 - 8000 psf |
| | | Hard/Very hard | Over 32 | > 8000 psf |

* Number of blows of 140 pound hammer falling 30" to drive a 2" O.D. (1-3/8" I.D.) split barrell sampler 1 foot (ASTM-1586 standard penetration test).

| | | | |
|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | SOIL CLASSIFICATION PROPERTIES LEGEND | M2 CONSULTANTS inc<br>638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | A-1A |
| CHECKED BY: A. JOHNS | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/17/2013 / APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 | | |

## ROCK TYPE

| | | | |
|---|---|---|---|
| CLAYSTONE OR MUDSTONE | SANDSTONE, MASSIVE, COARSE-GRAINED | BEDDED CHERT | MASSIVE IGNEOUS ROCK |
| SILTSTONE | SANDSTONE, BEDDED | TUFF AND TUFF-BRECCIA | META-VOLCANICS, GREENSTONE |
| SHALE | SANDSTONE, CROSS-BEDDED | RHYOLITE OR OTHER LAVA FLOWS | SERPENTINITE |
| SANDSTONE, MASSIVE, FINE-GRAINED | CONGLOMERATE | BASALT OR BASIC LAVA FLOWS | SCHIST |
| GREYWACKE | META SHALE | INTERBEDDED GREYWACKE & SHALE | TOPSOIL/COLLUVIUM |

## STRUCTURE

BEDDING/SPLITTING OF SEDIMENTARY ROCKS

| | |
|---|---|
| MASSIVE | 4.0 ft to no apparent bedding |
| THICK | 2.0 - 4.0 ft |
| MODERATE | 0.2 - 2.0 ft |
| THIN | 0.05 - 0.2 ft |
| SHALEY, PLATY, LAMINATED | 0.01 - 0.05 ft |
| PAPERY, THINLY LAMINATED | < 0.1 ft |

FRACTURING AND JOINTING INTENSITY

| | | |
|---|---|---|
| MA | MASSIVE | > 4.0 ft |
| SF | SLIGHTLY FRACTURED | 1.0 - 4.0 ft |
| MF | MODERATELY FRACTURED | 0.5 - 1.0 ft |
| CF | CLOSELY FRACTURED | 0.1 - 0.5 ft |
| IF | INTENSELY FRACTURED | 0.05 - 0.1 ft, no clay |
| CR | CRUSHED | all sizes, including clay |

## RECOVERY

- NO RECOVERY
- BULK SAMPLE
- CAL SAMPLER
- BULK SAMPLE
- SPT
- SHELBY TUBE
- MODIFIED CAL
- CAL BULK SAMPLE

## WEATHERING

DEGREE OF WEATHERING

| | | |
|---|---|---|
| FRW | FRESH | No evidence of mechanical or chemical alteration, no loss of strength |
| SW | SLIGHTLY WEATHERED | Slight discoloration on surface, < 10% altered, no loss of strength |
| MW | MODERATELY WEATHERED | Discoloring evident, 10 - 15% altered, no loss of strength |
| DW | DEEPLY WEATHERED | Entire mass discolored, minerals altered, fraction of original strength retained |
| VDW | DECOMPOSED | Residual soil, crumbled by hand |

## ROCK QUALITY DESCRIPTIONS

STRENGTH

| | | |
|---|---|---|
| VS | VERY STRONG | Difficult to fracture with repeated heavy hammer blows |
| S | STRONG | Withstands a few heavy hammer blows |
| MS | MODERATELY STRONG | Fractures with moderate hammer blows |
| WK | WEAK | Crumbles with light hammer blows |
| FR | FRIABLE | Crumbles easily in the fingers |
| PL | PLASTIC | Moldable |

HARDNESS

| | | |
|---|---|---|
| VH | VERY HARD | Cannot be scratched with a knife blade, leaves a metallic streak |
| H | HARD | Can be scratched with difficulty, scratch produces little powder and is only faintly visible |
| MH | MODERATELY HARD | Can be readily scratched with a knife, scratch leaves heavy trace of dust |
| LH | LOW HARDNESS | Can be gouged or scratched |
| FR | FRIABLE | Easily crumbled by fingers |
| PL | PLASTIC | Moldable |

ROCK QUALITY DESIGNATION (RQD)

| | | |
|---|---|---|
| E | EXCELLENT | 90 - 100% |
| G | GOOD | 75 - 90% |
| F | FAIR | 50 - 75% |
| P | POOR | 25 - 50% |
| VP | VERY POOR | < 25% |

| | | | |
|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | | SOIL CLASSIFICATION PROPERTIES LEGEND | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | A-1B |
| CHECKED BY: A. JOHNS | | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | |
| DATE: 01/17/2013 | APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 | |

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

| Hole No. | Date: 3/6/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30" drop, Auto Hammer |
|---|---|---|---|
| B-1 | Start Time: 10:30 | Crew: Vince / Angel | Drill: Fraste Mito - 40 Track Rig |
| | End Time: 16:00 | Inspector: Alastair Johns | Type of Bit: Drag bit / Tricone |

Surface Conditions: Grassy Meadow — Total Depth: 50' 2.5"

Location: 38° 24' 46.75" N, 122° 56' 36.02" W — Elevation: 635.56'

| Depth - ft | Recovery Method | Field Tests: Blows / ft | Field Tests: TV - tsf | Field Tests: Pen - tsf | Labratory Tests: Sample No. | Labratory Tests: Moisture Content % | Labratory Tests: Dry Density - pcf | Labratory Tests: Plasticity | Labratory Tests: Other Tests | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | 0.25 -1.3 | 2.5 | | | | LL 53 PI 33 | | | Silty clay w/ organics, CL, moist, 75 psi push pressure, [top soil] |
| | | | | | 1A | | | | Passing - #200 = 67.8% | | Silty clay with trace organics, CH, dark grey brown, very soft |
| 5 | | 2 | 0.4 | 2 - 2.5 | 3A | 30.3 | 91.6 | LL 54 PI 35 | Passing - #200 = 25.5% | | NO RECOVERY<br>Clayey sand, SC, dark greyish brown, wet, 100 psi push pressure, fine to coarse grained (alluvial deposits)<br>NO RECOVERY, soil consistency: loose to soft |
| | | 7 | | | | | | | | | Sandy fat clay, CH, dark grey brown, moist, soft |
| 10 | | 19 | 0.6 | 1.5 | 5A 5B 5C | 30.9 | 90.1 | LL 48 PI 27 | Passing - #200 = 48.9% Passing - #200 = 22.6% | | Clayey sand, SC, yellowyish brown, moist, stiff / medium dense, fine to coarse grained<br>Harder drilling at 13' for 12" - fine to coarse gravel in sample, then easier drilling |
| 15 | | 82 | | | 6A 6B | | | | | | Serpentinite bedrock, greenish grey, highly weathered, very closely fractured, weak to friable, soil consistency: dry, hard to very dense |
| 20 | | | | | | | | | | | |

| | | |
|---|---|---|
| COMPILED BY: A. Johns | **SOIL CLASSIFICATION PROPERTIES LEGEND** | M2 CONSULTANTS inc<br>638 19th Street, Unit 18<br>San Francisco, California 94107 |
| REVISED BY: A. Johns | | |
| CHECKED BY: Y. Prashar | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | |
| DATE: 3 / 20 / 2012 — APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO | |

Plate No.

A-2

Sheet: 1 of 3

| Hole No. | Date: 3/6/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30" drop, Auto Hammer |
|---|---|---|---|
| B-1 | Start Time: 10:30 | Crew: Vince / Angel | Drill: Fraste Mito - 40 Track Rig |
| | End Time: 16:00 | Inspector: Alastair Johns | Type of Bit: Drag bit / Tricone |

Surface Conditions: Grassy Meadow — Total Depth: 50' 2.5"

Location: 38° 24' 46.75" N, 122° 56' 36.02" W — Elevation: 635.56'

| Depth - ft | Recovery Method | Field Tests: Blows / ft | Field Tests: TV - tsf | Field Tests: Pen - tsf | Labratory Tests: Sample No. | Labratory Tests: Moisture Content % | Labratory Tests: Dry Density - pcf | Labratory Tests: Plasticity | Labratory Tests: Other Tests | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | | | | | | | | | | dark grey, moderately to highly weathered, weak |
| | | >75 | | | 7A | | | | | | |
| | | | | | | | | | | | Hard drilling from 23' |
| 25 | | | | | | | | | | | |
| 30 | | | | | | | | | | | bluish grey, slightly weathered, weak to moderately strong. |
| | | >85 | | | 8A | | | | | | |
| 35 | | | | | | | | | | | |
| 40 | | | | | | | | | | | |

| | | |
|---|---|---|
| COMPILED BY: A. Johns | SOIL CLASSIFICATION PROPERTIES LEGEND | Plate No. |
| REVISED BY: A. Johns | | A-2 |
| CHECKED BY: Y. Prashar | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | |
| DATE: 3 / 20 / 2012 — APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO | Sheet: 2 of 3 |

M2 CONSULTANTS inc
638 19th Street, Unit 18
San Francisco, California 94107

| Hole No. | Date: 3/6/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30" drop, Auto Hammer |
|---|---|---|---|
| B-1 | Start Time: 10:30 | Crew: Vince / Angel | Drill: Fraste Mito - 40 Track Rig |
| | End Time: 16:00 | Inspector: Alastair Johns | Type of Bit: Drag bit / Tricone |

Surface Conditions: Grassy Meadow — Total Depth: 50' 2.5"

Location: 38° 24' 46.75" N, 122° 56' 36.02" W — Elevation: 635.56'

| Depth - ft | Recovery Method | Field Tests: Blows / ft | Field Tests: TV - tsf | Field Tests: Pen - tsf | Labratory Tests: Sample No. | Labratory Tests: Moisture Content % | Labratory Tests: Dry Density - pcf | Labratory Tests: Plasticity | Labratory Tests: Other Tests | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 40 | | 50/2" | | | 9A | | | | | | |
| 45 | | | | | | | | | | | |
| 50 | | 50/2.5" | | | 10A | | | | | | Boring terminated at ~ 50' 2.5"<br>Boring grouted after completion |
| 55 | | | | | | | | | | | |
| 60 | | | | | | | | | | | |

| | | |
|---|---|---|
| COMPILED BY: A. Johns | SOIL CLASSIFICATION PROPERTIES LEGEND | M2 CONSULTANTS inc<br>638 19th Street, Unit 18<br>San Francisco, California 94107 |
| REVISED BY: A. Johns | | |
| CHECKED BY: Y. Prashar | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | |
| DATE: 3 / 20 / 2012 — APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO | |

Plate No.

A-2

Sheet: 3 of 3

| Hole No. | Date: 3/7/12 | Drilling Agency: V & W | Hammer Data: 140 lbs, 30" drop Auto Hammer |
|---|---|---|---|
| B-2 | Start Time: 7:00 | Crew: Vince / Angel | Drill: Fraste Mito - 40 Track Rig |
| | End Time: 10:15 | Inspector: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Grassy Meadow — Total Depth: 31.5'

Location: 38° 24' 45.39" N, 122° 56' 31.83" W — Elevation: 646.48'

| Depth - ft | Recovery Method | Field Tests: Blows / ft | Field Tests: TV - tsf | Field Tests: Pen - tsf | Labratory Tests: Sample No. | Moisture Content % | Dry Density - pcf | Plasticity | Other Tests | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | | | Silty clay with organics, OH, [top soil] |
| | | 2/4" | 0.4 | 1 | 1A | 27.7 | 95.5 | LL 46 PI 28 | Passing - #200 = 52.2% | | Silty clay w/ trace sand, CL, dark brown, moist, 50 psi push pressure, medium to coarse grained, ~14" recovery |
| 5 | | 12 | 0.35 - 0.45 | 1 - 2 | 2B<br>2C | 22.8 | 100 | | | | Sandy clay, CL, dark brown, moist, stiff, fine sand |
| | | | 0.5 | 1 | 3A | | | LL 53 PI 33 | | | Silty clay w/ gravel, CH, redish brown, moist, 65 psi push pressure, medium to coarse gravel |
| 10 | | 10 | 0.45 | 0.5 - 1 | 4B<br>4C<br>5B | 46.1 | 72 | LL 69 PI 46 | Passing - #200 = 37.4% | | Clayey sand, SC, redish brown, moist, medium stiff to loose, fine to coarse sand<br>Clayey sand w/ sand, SC, yellowish red, hard to very dense, coarse gravel |
| 15 | | >63<br>38 | | | 5C<br>6A | | | | | | Sheared serpentinite bedrock BR at 11'<br>Serpentinite bedrock, light green, highly weathered, very closely fractured, friable, soil consistency: dry, very stiff to dense |
| 20 | | | | | | | | | | | |

| | | |
|---|---|---|
| COMPILED BY: A. Johns | **SOIL CLASSIFICATION PROPERTIES LEGEND** | Plate No. |
| REVISED BY: A. Johns | | A-3 |
| CHECKED BY: Y. Prashar | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | |
| DATE: 3 / 20 / 2012 — APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO | Sheet: 1 of 2 |

M2 CONSULTANTS inc
638 19th Street, Unit 18
San Francisco, California 94107

| Hole No. | Date: 3/7/12 | Drilling Agency: V & W | Hammer Data: 140 lbs, 30" drop Auto Hammer |
|---|---|---|---|
| B-2 | Start Time: 7:00 | Crew: Vince / Angel | Drill: Fraste Mito - 40 Track Rig |
| | End Time: 10:15 | Inspector: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Grassy Meadow — Total Depth: 31.5'

Location: 38° 24' 45.39" N, 122° 56' 31.83" W — Elevation: 646.48'

| Depth - ft | Recovery Method | Field Tests: Blows / ft | Field Tests: TV - tsf | Field Tests: Pen - tsf | Labratory Tests: Sample No. | Labratory Tests: Moisture Content % | Labratory Tests: Dry Density - pcf | Labratory Tests: Plasticity | Labratory Tests: Other Tests | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | 24 | | | 7A | | | | | | greenish grey, friable to plastic, soil consistency: very stiff to medium dense |
| 25 | | | | | | | | | | | light to dark green, soil consistency: hard to dense |
| 30 | | 49 | | | 8A | | | | | | |
| | | | | | | | | | | | Boring terminated at ~31.5'<br>Boring grouted after completion |
| 35 | | | | | | | | | | | |
| 40 | | | | | | | | | | | |

| | | |
|---|---|---|
| COMPILED BY: A. Johns | **SOIL CLASSIFICATION PROPERTIES LEGEND** | M2 CONSULTANTS inc<br>638 19th Street, Unit 18<br>San Francisco, California 94107 |
| REVISED BY: A. Johns | | |
| CHECKED BY: Y. Prashar | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | |
| DATE: 3 / 20 / 2012 — APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO | |

Plate No.

A-3

Sheet: 2 of 2

| Hole No. | Date: 2/22/12 | Drilling Agency: V & W | Hammer Data: 140 lbs, 30" drop Auto Hammer |
|---|---|---|---|
| B-3 | Start Time: 12:30 | Crew: Frank | Drill: CME 75, Auto Hammer |
| | End Time: 16:15 | Inspector: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Grassy Meadow — Total Depth: 19'

Location: 38.413702, -122.941976 — Elevation: 647

| Depth - ft | Recovery Method | Field Tests: Blows / ft | Field Tests: TV - tsf | Field Tests: Pen - tsf | Labratory Tests: Sample No. | Moisture Content % | Dry Density - pcf | Plasticity | Other Tests | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | | | Silty organic clay, OH, humic dark brown, [top soil] |
| | | | 1.5 | | | | | | | | Silty fat clay, SM/CH, grey brown, moist, medium stiff |
| | | 9 | | | 1C | 29.5 | 91.7 | LL 71 PI 42 | | | Sandy clay, SC, dark greyish brown, moist, stiff, fine to medium grained sand |
| 5 | | | 1.3 | 2.5 | 2B | | | LL 66 PI 38 | Passing - #200 = 50% | | |
| | | 16 | | | 2C | | | | | | |
| | | >90 | | | 3C | 8.2 | 129 | | Passing - #200 = 10.7% | | Serpentinite bedrock, olive brown, completely weathered, friable to plastic, fine to coarse grained, iron oxide stain, mantle layers, sheared clasts |
| | | | | | | | | | | | change to HQ coring at 10' |
| 10 | | >87 | | | 4C | | | | | | olive brown, granular and fibrous, fine sandstone texture in matrix, 250 psi push pressure, drilled 24" in 20 minutes, void in sample from 10' - 11.5' |
| | | | | | 5A | | | | | | olive-yellow / dark brown-green, foliated, 100% recovery |
| | | | 0.63 | 4 - 4.5 | | | | | | | |
| 15 | | | | | 6A | | | | | | light to medium green, granular, soil consistency: CH fat clay, 500 psi push pressure |
| | | | 0.63 - 2.5 | 4 - 4.5 | 7A | | | | | | Sheared Meta-shale, dark grey, slightly weathered, moderately fractured, very strong, quartz veins, lightly drilled into knocker |
| 20 | | | | | | | | | | | Breccia, greyish dark brown, moderately weathered, moderately fractured, moderately strong |

| | | |
|---|---|---|
| COMPILED BY: A. Johns | SOIL CLASSIFICATION PROPERTIES LEGEND | M2 CONSULTANTS inc |
| REVISED BY: A. Johns | | 638 19th Street, Unit 18 |
| CHECKED BY: Y. Prashar | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | San Francisco, California 94107 |
| DATE: 3 / 20 / 2012 — APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO | |

Plate No.

A-4

Sheet: 1 of 2

| Hole No. | Date: 2/22/12 | Drilling Agency: V & W | Hammer Data: 140 lbs, 30" drop Auto Hammer |
|---|---|---|---|
| B-3 | Start Time: 12:30 | Crew: Frank | Drill: CME 75, Auto Hammer |
| | End Time: 16:15 | Inspector: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Grassy Meadow — Total Depth: 19'

Location: 38.413702, -122.941976 — Elevation: 647

| Depth - ft | Recovery Method | Field Tests: Blows / ft | Field Tests: TV - tsf | Field Tests: Pen - tsf | Labratory Tests: Sample No. | Labratory Tests: Moisture Content % | Labratory Tests: Dry Density - pcf | Labratory Tests: Plasticity | Labratory Tests: Other Tests | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | | | | | | | | | | Boring terminated at ~19' |
| | | | | | | | | | | | Boring grouted after completion |
| 25 | | | | | | | | | | | |

| | | |
|---|---|---|
| COMPILED BY: A. Johns | **SOIL CLASSIFICATION PROPERTIES LEGEND** | M2 CONSULTANTS inc<br>638 19th Street, Unit 18<br>San Francisco, California 94107 |
| REVISED BY: A. Johns | | |
| CHECKED BY: Y. Prashar | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | |
| DATE: 3 / 20 / 2012 — APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO | |

Plate No.

A-4

Sheet: 2 of 2

| Hole No. | Date: 2/23/12 | Drilling Agency: V & W | Hammer Data: 140 lbs, 30" drop Auto Hammer |
|---|---|---|---|
| B-4 | Start Time: 7:30 | Crew: Frank | Drill: CME75 |
| | End Time: 10:00 | Inspector: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Grassy Meadow — Total Depth: 16.5'

Location: 38.41335, -122.942632 — Elevation: 636'

| Depth - ft | Recovery Method | Field Tests: Blows / ft | Field Tests: TV - tsf | Field Tests: Pen - tsf | Labratory Tests: Sample No. | Moisture Content % | Dry Density - pcf | Plasticity | Other Tests | USCS | Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | 0 | | | | | | | Organic silt and clay, OH/CL, very soft, [top soil] |
| | | 13 | 0.75 - 0.8 | 2.5 | 1B 1C | | | | | | Silty fat clay, CH/CL, dark greyish brown, moist, stiff |
| | | | | | | | | | | | Silty fat clay, CH, dark greyish brown, moist, very stiff, with some sand, medium grained |
| | | 26 | | | 2B 2C | | | | | | |
| 5 | | | | | | | | | | | w/ weathered sandstone sand, medium grained |
| | | 23 | 1.7 | 3.5 - 4 | 3B 3C | | | | | | |
| | | | | | | | | | | | medium stiff to stiff, w/ sandstone sand, coarse grained |
| | | 11 | 0.6 | 1 | 4B 4C | | | | | | |
| | | | | | | | | | | | Clayey sand, SC, red brown, very loose to loose, fine to medium grained |
| 10 | | 5 | 0.3 - 0.35 | 0.5 | 5B 5C | 39.1 | 81 | LL 57 PI 38 | Passing - #200 = 41.7% | | |
| | | | | | | | | | | | Harder drilling at 12' |
| | | | | | | | | | | | Serpentinite bedrock, olive / light dark green, highly weathered, very closely fractured and sheared, friable to plastic, variable texture |
| 15 | | 49 | | | 6A | | | | | | |
| | | | | | | | | | | | Boring terminated at ~16.5' |
| | | | | | | | | | | | Boring grouted after completion |
| 20 | | | | | | | | | | | |

| | | |
|---|---|---|
| COMPILED BY: A. Johns | SOIL CLASSIFICATION PROPERTIES LEGEND | Plate No. |
| REVISED BY: A. Johns | | A-5 |
| CHECKED BY: Y. Prashar | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | |
| DATE: 3 / 20 / 2012 — APPROVED BY: Y. Prashar | PROJECT NO: 01202/GEO | Sheet: 1 of 1 |

M2 CONSULTANTS inc
638 19th Street, Unit 18
San Francisco, California 94107

# APPENDIX A

# SITE 3C-2

# BORING LEGEND &

# BORING LOGS B-5 THROUGH B-13

| MAJOR DIVISIONS | | | SYMBOLS | SECONDARY DIVISIONS |
|---|---|---|---|---|
| COARSE GRAINED SOILS<br>More than half is larger than No. 28 sieve | GRAVELS<br>More than half of coarse fraction retained on No. 4 sieve | CLEAN GRAVELS (Little or no fines) | GW | Well-graded gravels, gravel-sand mixtures, little or no fines. |
| | | | GP | Poorly-graded gravels, gravel-sand mixtures, little or no fines. |
| | | GRAVELS WITH FINES (Appreciable amount of fines) | GM | Silty gravels, gravel-sand-silt mixtures. |
| | | | GC | Clayey gravels, gravel-sand-clay mixtures. |
| | SANDS<br>More than half of coarse fraction passing on No. 4 sieve | CLEAN SANDS (Little or no fines) | SW | Well-graded sands, gravelly sands, little or no fines. |
| | | | SP | Poorly-graded sands, gravelly sands, little or no fines. |
| | | SANDS WITH FINES (Appreciable amount of fines) | SM | Silty sands, sand-silt mixtures. |
| | | | SC | Clayey sands, sand-clay mixtures. |
| FINE GRAINED SOILS<br>More than half is smaller than No. 200 sieve | SILTS AND CLAYS | Liquid Limit less than 50 | ML | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity. |
| | | | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. |
| | | | OL | Organic silts and organic silty clays of low plasticity. |
| | SILTS AND CLAYS | Liquid Limit greater than 50 | MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts. |
| | | | CH | Inorganic clays of high plasticity, fat clays. |
| | | | OH | Organic clays of medium to high plasticity, organic silts. |
| HIGHLY ORGANIC SOILS | | | PT | Peat and other highly organic soils. |

Note: Dual symbols are used to indicate borderline soil classifications

## GRAIN SIZE

U.S. STANDARD SERIES SIEVE

| | #200 | #40 | #10 | #4 | 3/4" | 3" | 12" | |
|---|---|---|---|---|---|---|---|---|
| SILTS AND CLAYS DISTINGUISHED ON BASIS OF PLASTICITY | SAND | | | GRAVEL | | COBBLES | BOULDERS | |
| | FINE | MEDIUM | COARSE | FINE | COARSE | | | |
| | 0.075 mm | 0.475 mm | 2.00 mm | 4.75 mm | 19 mm | 75 mm | 300 mm | |

## PENETRATION RESISTANCE

| SANDS AND GRAVELS | | CLAYS AND SILTS | | |
|---|---|---|---|---|
| IN SITU DENSITY | SPT BLOWS/FOOT* | CONSISTENCY | SPT BLOWS/FOOT* | POCKET PENETROMETER |
| Very loose | 0 - 4 | Very Soft | 0 - 2 | 0 - 500 psf |
| Loose | 4 - 10 | Soft | 2 - 4 | 500 - 1000 psf |
| Medium dense | 10 - 30 | Firm/Medium stiff | 4 - 8 | 1000 - 2000 psf |
| Dense | 30 - 50 | Stiff | 8 - 16 | 2000 - 4000 psf |
| Very dense | Over 50 | Very Stiff | 16 - 32 | 4000 - 8000 psf |
| | | Hard/Very hard | Over 32 | > 8000 psf |

* Number of blows of 140 pound hammer falling 30" to drive a 2" O.D. (1-3/8" I.D.) split barrell sampler 1 foot (ASTM-1586 standard penetration test).

| | | | |
|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | SOIL CLASSIFICATION PROPERTIES LEGEND | M2 CONSULTANTS inc<br>638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | A-1A |
| CHECKED BY: A. JOHNS | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/17/2013 / APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 | | |

## ROCK TYPE

| | | | |
|---|---|---|---|
| CLAYSTONE OR MUDSTONE | SANDSTONE, MASSIVE, COARSE-GRAINED | BEDDED CHERT | MASSIVE IGNEOUS ROCK |
| SILTSTONE | SANDSTONE, BEDDED | TUFF AND TUFF-BRECCIA | META-VOLCANICS, GREENSTONE |
| SHALE | SANDSTONE, CROSS-BEDDED | RHYOLITE OR OTHER LAVA FLOWS | SERPENTINITE |
| SANDSTONE, MASSIVE, FINE-GRAINED | CONGLOMERATE | BASALT OR BASIC LAVA FLOWS | SCHIST |
| GREYWACKE | META SHALE | INTERBEDDED GREYWACKE & SHALE | TOPSOIL/COLLUVIUM |

## STRUCTURE

BEDDING/SPLITTING OF SEDIMENTARY ROCKS

| | |
|---|---|
| MASSIVE | 4.0 ft to no apparent bedding |
| THICK | 2.0 - 4.0 ft |
| MODERATE | 0.2 - 2.0 ft |
| THIN | 0.05 - 0.2 ft |
| SHALEY, PLATY, LAMINATED | 0.01 - 0.05 ft |
| PAPERY, THINLY LAMINATED | < 0.1 ft |

FRACTURING AND JOINTING INTENSITY

| | | |
|---|---|---|
| MA | MASSIVE | > 4.0 ft |
| SF | SLIGHTLY FRACTURED | 1.0 - 4.0 ft |
| MF | MODERATELY FRACTURED | 0.5 - 1.0 ft |
| CF | CLOSELY FRACTURED | 0.1 - 0.5 ft |
| IF | INTENSELY FRACTURED | 0.05 - 0.1 ft, no clay |
| CR | CRUSHED | all sizes, including clay |

## RECOVERY

- NO RECOVERY
- BULK SAMPLE
- CAL SAMPLER
- BULK SAMPLE
- SPT
- SHELBY TUBE
- MODIFIED CAL
- CAL BULK SAMPLE

## WEATHERING

DEGREE OF WEATHERING

| | | |
|---|---|---|
| FRW | FRESH | No evidence of mechanical or chemical alteration, no loss of strength |
| SW | SLIGHTLY WEATHERED | Slight discoloration on surface, < 10% altered, no loss of strength |
| MW | MODERATELY WEATHERED | Discoloring evident, 10 - 15% altered, no loss of strength |
| DW | DEEPLY WEATHERED | Entire mass discolored, minerals altered, fraction of original strength retained |
| VDW | DECOMPOSED | Residual soil, crumbled by hand |

## ROCK QUALITY DESCRIPTIONS

STRENGTH

| | | |
|---|---|---|
| VS | VERY STRONG | Difficult to fracture with repeated heavy hammer blows |
| S | STRONG | Withstands a few heavy hammer blows |
| MS | MODERATELY STRONG | Fractures with moderate hammer blows |
| WK | WEAK | Crumbles with light hammer blows |
| FR | FRIABLE | Crumbles easily in the fingers |
| PL | PLASTIC | Moldable |

HARDNESS

| | | |
|---|---|---|
| VH | VERY HARD | Cannot be scratched with a knife blade, leaves a metallic streak |
| H | HARD | Can be scratched with difficulty, scratch produces little powder and is only faintly visible |
| MH | MODERATELY HARD | Can be readily scratched with a knife, scratch leaves heavy trace of dust |
| LH | LOW HARDNESS | Can be gouged or scratched |
| FR | FRIABLE | Easily crumbled by fingers |
| PL | PLASTIC | Moldable |

ROCK QUALITY DESIGNATION (RQD)

| | | |
|---|---|---|
| E | EXCELLENT | 90 - 100% |
| G | GOOD | 75 - 90% |
| F | FAIR | 50 - 75% |
| P | POOR | 25 - 50% |
| VP | VERY POOR | < 25% |

| | | | |
|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | | SOIL CLASSIFICATION PROPERTIES LEGEND | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | A-1B |
| CHECKED BY: A. JOHNS | | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | |
| DATE: 01/17/2013 | APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 | |

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

| Boring No. | Date: 11/09/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| B-5 | Start Time: 1045 | Crew: Robert / Chad | Drill: B-53 Truck Rig |
| | End Time: 1430 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Grassy pasture — Total Depth: 58 feet

Location: SITE 3C-2 38°24'38.75"N, 122°56'10.33"W — Elevation: 890 feet amsl

| Depth | Recovery Method | Field Tests: Blow / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Moisture Conetent % | Dry Unit Weight - pcf | Atterberg Limits | Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | | | |
| 5 | | 52 | 3.5 | 2.0 | 1B<br>1C | | | LL 38<br>PI 11 | TXUU | | Sandstone CONGLOMERATE, red-brown to yellow to dark brown, completely weathered, mostly plastic with some friable, [dense], clasts are generally subangular |
| 10 | | 42 | 3.5 | 4.5 | 2B<br>2C | | | LL 39<br>PI 12 | | | Sandstone CONGLOMERATE, variable clast colors, completely weathered, friable to plastic, [dense] |
| 15 | | >91 | | | 3B<br>3C | | | | Passing - #200 = 20.2% TXUU | | Sandstone CONGLOMERATE, variable clast colors, sandstone matrix is highly weathered, friable, [very dense], free water observed in sample |
| 20 | | | | | | | | | | | |

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-6 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAI, CALIFORNIA | | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | 638 19th Street, Unit 18 San Francisco, California 94107 | Sheet: 1 of 3 |

| Boring No. B-5 | Date: 11/09/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 1045 | Crew: Robert / Chad | Drill: B-53 Truck Rig |
| | End Time: 1430 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Grassy pasture — Total Depth: 58 feet

Location: SITE 3C-2 38°24'38.75"N, 122°56'10.33"W — Elevation: 890 feet amsl

| Depth | Recovery Method | Field Tests: Blow / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | >84 | | | 4B | | | | | | Sandstone CONGLOMERATE, mostly oxidized red, sandstone matrix is mostly pale brown, highly weathered, mostly friable [very dense], water rapidly seeped into formation |
| | | | | | 4C | | | LL 42 PI 14 | | | |
| 25 | | 50/4" | | | 5A | | | | | | SHALE, dark bluish grey, highly weathered, friable, [very dense] |
| 30 | | 50/5" | | | | | | | | | [Weak] |
| | | | | | 6A | | | | | | GREYWACKE, dark bluish grey, moderately weathered, moderately fractured, [weak to moderately strong], consists of poorly sorted fine to coarse sandstone matrix with 1-2 inch subangular sandstone clasts [very dense], contains quartz veins and horizons of shale and strong meta shale |
| | | 20% | | | 7A | | | | | | |
| 35 | | | | | | | | | | | |
| | | 27% | | | 8A | | | | | | GREYWACKE, dark bluish grey, moderately weathered,moderately fractured, [weak], horizons of strong meta shale, shale, and greywacke sandstone, sandstone matrix horizons are moderately well sorted, horizons with clasts are poorly sorted |
| 40 | | | | | | | | | | | |

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-6 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAI, CALIFORNIA | 638 19th Street, Unit 18 San Francisco, California 94107 | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | | Sheet: 2 of 3 |

| Boring No. B-5 | Date: 11/09/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 1045 | Crew: Robert / Chad | Drill: B-53 Truck Rig |
| | End Time: 1430 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Grassy pasture — Total Depth: 58 feet

Location: SITE 3C-2 38°24'38.75"N, 122°56'10.33"W — Elevation: 890 feet amsl

| Depth | Recovery Method | Field Tests: Blow / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 40 | | | | | | | | | | | |
| | | 20% | | | 9A | | | | | | Rythmic horizons of weak grey SHALE and moderately sorted GREYWACKE, highly to moderately weathered, closely fractured, sandstone is [moderately strong], shale is [weak] |
| 45 | | | | | | | | | | | |
| | | 22.5% | | | 10A | | | | | | GREYWACKE, bluish grey, moderately weathered, moderately fractured, shale horizons at approximately 49 feet, persistent sub vertical fractures |
| 50 | | | | | | | | | | | |
| | | 7% | | | 11A | | | | | | SHALE, dark bluish grey, moderately weathered, moderately fractured, [weak and plastic], numerous sub vertical fractures, shale too weak to core, bag sample of sand sized shale |
| 55 | | | | | | | | | | | |
| | | 0% | | | 12A | | | | | | |

END OF BORING AT 58 FEET

Temporary piezometer installed at 33' on 11/13/2012, Groundwater measurements taken on 11/14/12 at 0745 = 14 feet, on 11/14/12 at 1310 = 15 feet, on 11/14/12 at 1600 = 15.5 feet, stabilized groundwater at 15.5 feet

Temporary piezometer removed and boring grouted using tremi pipe to existing ground surface

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-6 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAI, CALIFORNIA | | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | 638 19th Street, Unit 18 San Francisco, California 94107 | Sheet: 3 of 3 |

| Boring No. B-6 | Date: 11/08/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 1300 | Crew: Robert / Luis | Drill: B-53 Truck Rig |
| | End Time: 0930 | Geologist: Alastair Johns | Type of Bit: Sundox |

Surface Conditions: Generally flat forested area — Total Depth: 60 feet

Location: SITE 3C-2 38°24'39.56"N, 122°56'11.06"W — Elevation: 890 feet amsl

| Depth | Recovery Method | Field Tests: Blow / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | | | |
| 5 | | 38 | | 4.50 | 1B<br>1C | | | | | | Sandstone CONGLOMERATE, sandstone matrix pale brown, clasts red to pink to brown, highly weathered, friable, [medium dense], clasts appear to be sandstone, matrix is fine to coarse grained, subangular grains, poorly sorted, 1-2 inch subangular shale clasts and <1 inch subrounded meta shale pebbles |
| 10 | | 27 | | 4.50 | 2B<br>2C | | | LL 41<br>PI 13 | | | Same, appears to have more mafic clasts (darker-igneous), matrix fine to medium grained pink sandstone, increasingly well sorted, [medium dense] |
| 15 | | 40 | | 4.25 | 3B<br>3C | | | | | | Same, [medium dense to dense], fine to coarse grained poorly sorted subrounded tuff pebbles <0.5 inch, completely weathered subangular shale remnants <1 inch, pocket penetrometer in matrix |
| 20 | | | | | | | | | | | |

COMPILED BY: N. Scheemaker
REVISED BY: N. Scheemaker
CHECKED BY: A. Johns
DATE: 01/17/2013
APPROVED BY: Y. Prashar

**BORING LOG**

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / Geo 2

M2 CONSULTANTS inc
638 19''' Street, Unit 18
San Francisco, California 94107

Plate No. A-7

Sheet: 1 of 3

| Boring No. B-6 | Date: 11/08/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 1300 | Crew: Robert / Luis | Drill: B-53 Truck Rig |
| | End Time: 0930 | Geologist: Alastair Johns | Type of Bit: Sundox |

Surface Conditions: Generally flat forested area — Total Depth: 60 feet

Location: SITE 3C-2 38°24'39.56"N, 122°56'11.06"W — Elevation: 890 feet amsl

| Depth | Recovery Method | Field Tests: Blow / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | 58 | | | 4B | | | | TXUU | | Same, [dense], increasingly poorly sorted |
| | | | | | 4C | | | | | | |
| 25 | | 57 | | | 5B | | | | | | Same, [dense], increasing proportion of dark grey to brown subrounded >0.5 inch, strong tuffaceous pebbles |
| | | | | | 5C | | | | | | |
| 30 | | 76 | | | 6B | | | | | | Same, [very dense], poorly sorted |
| | | | | | 6C | | | | TXUU | | |
| 35 | | >90 | | | 7B | | | | | | CONGLOMERATE, [very dense], consists of sandstone and tuffaceous horizons, poorly cemented fine grained tuffaceous pea sized gravel (GW), heterogeneous, gravel is angular, similar horizons also present in B-7 |
| | | | | | 7C | | | | | | |
| 40 | | | | | | | | | | | |

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-7 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18 San Francisco, California 94107 | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | | Sheet: 2 of 3 |

| Boring No. B-6 | Date: 11/08/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 1300 | Crew: Robert / Luis | Drill: B-53 Truck Rig |
| | End Time: 0930 | Geologist: Alastair Johns | Type of Bit: Sundox |

Surface Conditions: Generally flat forested area — Total Depth: 60 feet

Location: SITE 3C-2 38°24'39.56"N, 122°56'11.06"W — Elevation: 890 feet amsl

| Depth | Recovery Method | Field Tests: Blow / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 40 | | 77 | | | 8B | | | | | | Poorly cemented and loose gravel (GW), [very dense], same as above, poorly sorted sandstone and tuffaceous gravel |
| | | | | | 8C | | | | | | |
| | | | | | | | | | | | META SHALE, greenish grey, highly weathered, plastic to strong, [very dense] |
| 45 | | >72 | | | | | | | | | No recovery, grey, highly weathered, [very dense], clayey gravel consistency material |
| | | | | | | | | | | | Harder drilling at 48 feet |
| 50 | | 64 | | | 9B | | | | | | META SHALE, dark bluish grey, slightly weathered, very strong, [dense], ground up in sampler, appears fresh |
| | | | | | | | | | | | Rythmic layers of SHALE and GREYWACKE, greywacke is moderately sorted, mostly fine grained |
| | | 33% | | | 10A | | | | | | GREYWACKE, dark greenish grey, slightly weathered to fresh, closely to moderately fractured (4-8 inches), moderately strong |
| 55 | | 50% | | | 11A | | | | | | |
| | | 33% | | | 12A | | | | | | GREYWACKE, closely fractured, remnants of quartz veins on fracture faces, meta shale is very strong, mostly fresh |
| 60 | | | | | | | | | | | END OF BORING AT 60 FEET, Boring grouted using tremi pipe to existing ground surface |

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-7 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | 638 19"' Street, Unit 18 San Francisco, California 94107 | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | | Sheet: 3 of 3 |

| Boring No. B-7 | Date: 11/07/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 1600 | Crew: Robert / Luis | Drill: B-53 Truck Rig |
| | End Time: 1130 | Inspector: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Flat soil area — Total Depth: 56 feet

Location: SITE 3C-2 38°24'40.67"N, 122°56'11.24"W — Elevation: 885 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | | | |
| 5 | | 33 | | >4.5 | 1B<br>1C | | | | | | CONGLOMERATE, reddish yellow fine grained angular sandstone matrix pale green shale clasts >2 inches, highly weathered, friable plastic, [medium dense] |
| 10 | | 42 | | >4.5 | 2B<br>2C | | | LL 39<br>PI 11 | TXUU | | CONGLOMERATE, yellowish red, highly weathered, friable, [dense], mostly fine grained poorly sorted sandstone matrix with primarily sub rounded sandstone clasts |
| 15 | | 37 | | 4.25 | 3B<br>3C | | | | | | CONGLOMERATE, variable colors but primarily brown, highly weathered, friable, fine grained sandstone matrix, predominantly subangular, poorly sorted sandstone, shale, and meta shale clasts, [medium dense] |
| 20 | | | | | | | | | | | |

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-8 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18 San Francisco, California 94107 | |
| DATE: 01/17/2013 — APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | | Sheet: 1 of 3 |

| Boring No. B-7 | Date: 11/07/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 1600 | Crew: Robert / Luis | Drill: B-53 Truck Rig |
| | End Time: 1130 | Inspector: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Flat soil area — Total Depth: 56 feet

Location: SITE 3C-2 38°24'40.67"N, 122°56'11.24"W — Elevation: 885 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Moisture Conetent % | Dry Unit Weight - pcf | Atterberg Limits | Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | 74 | | 4.25 | 4B | | | | | | Same, [very dense] |
| | | | | | 4C | | | | TXUU | | |
| 25 | | 72 | | | 5B | | | | | | CONGLOMERATE, mostly sandstone, dark yellowish brown to reddish yellow, highly weathered, clasts vary in degree of weathering, in some areas weathered to clayey consistency, mostly friable, some plastic, clasts are sandstone, shale, and subrounded tuff, tuffaceous clasts are very strong, [very dense] |
| | | | | | 5C | | | | | | |
| 30 | | 50/6" | | | 6A | | | | | | Meta greywacke CONGLOMERATE, very dark brown where weathered, greenish grey when slightly weathered, poorly sorted, appears crystalline, meta shale clasts are highly weathered, [very dense] |
| | | 8% | | | 7A | | | | | | Sandstone CONGLOMERATE, dark greenish grey when moderately weathered, dark reddish brown when highly weathered, friable to weak, medium grained, clasts are shale, meta shale and sandstone |
| 35 | | 0% | | | 8A | | | | | | No recovery, cuttings in mud pan indicate that the bedrock is still conglomeritic |
| 40 | | | | | | | | | | | |

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-8 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18 San Francisco, California 94107 | |
| DATE: 01/17/2013 — APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | | Sheet: 2 of 3 |

| Boring No. B-7 | Date: 11/07/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 1600 | Crew: Robert / Luis | Drill: B-53 Truck Rig |
| | End Time: 1130 | Inspector: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Flat soil area — Total Depth: 56 feet

Location: SITE 3C-2 38°24'40.67"N, 122°56'11.24"W — Elevation: 885 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Torvane - tsf | Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Moisture Content % | Dry Unit Weight - pcf | Atterberg Limits | Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 40 | | 56 | | | 9B | | | | Passing - #200 = 0.0% | | CONGLOMERATE, many different colors, fine gravel sized clasts, [dense] |
| | | | | | 9C | | | | | | |
| 45 | | 74 | | | 10B | | | | | | GREYWACKE, completely weathered, grey, friable and plastic, mostly fine grained and well sorted, [very dense] |
| | | | | | 10C | | | | | | |
| 50 | | >86 | | | 11C | | | | | | SHALE, grey, highly to completely weathered, friable, [very dense] |
| 55 | | >90 | | | 12B | | | | | | Same, [very dense] |

END OF BORING AT 56 FEET

Temporary piezometer installed at 56' on 11/08/2012, Groundwater measurements taken, on 11/08/12 at 1545 = 21 feet, on 11/09/12 at 0720 = 20 feet, on 11/09/12 at 0945 = 20.5 feet, stabilized at 20.5 feet

Temporary piezometer removed and boring grouted using tremi pipe to existing ground surface

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-8 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | 638 19th Street, Unit 18 San Francisco, California 94107 | Sheet: 3 of 3 |

| Boring No. | Date: 11/12/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| B-8 | Start Time: 1545 | Crew: Luis / Nick | Drill: B-53 Truck Rig |
| | End Time: 1030 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Grassy pasture — Total Depth: 38 feet

Location: SITE 3C-2 38°24'39.25"N, 122°56'09.36"W — Elevation: 886 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Moisture Content % | Dry Unit Weight - pcf | Atterberg Limits | Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | | | |
| 5 | | 51 | 5.5 | >4.5 | 1B<br>1C | | | LL 44<br>PI 16 | | | CONGLOMERATE, yellowish red, pinkish, and white, highly weathered, friable and plastic, [dense], pale green subrounded shale clasts |
| 10 | | 29 | | | 2C | 16 | 106 | | Passing - #200 = 19.1% | | CONGLOMERATE, clasts of various colors, highly weathered, friable, [medium dense], subrounded 0.5-1 inches dark grey to brown tuff pebbles in sample<br>CONGLOMERATE, reddish yellow, fine grained matrix with subangular and subrounded clasts of sandstone, shale, meta shale, and chert |
| 15 | | 42 | | | 3B<br>3C | | | | | | CONGLOMERATE, various colors, highly weathered, friable, poorly sorted, clasts of sandstone, shale, and meta shale, [dense] |
| 20 | | | | | | | | | | | |

COMPILED BY: N. Scheemaker
REVISED BY: N. Scheemaker
CHECKED BY: A. Johns
DATE: 01/17/2013
APPROVED BY: Y. Prashar

# BORING LOG

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / Geo 2

M2 CONSULTANTS inc
638 19th Street, Unit 18
San Francisco, California 94107

Plate No.
A-9
Sheet: 1 of 2

| Boring No. B-8 | Date: 11/12/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 1545 | Crew: Luis / Nick | Drill: B-53 Truck Rig |
| | End Time: 1030 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Grassy pasture — Total Depth: 38 feet

Location: SITE 3C-2 38°24'39.25"N, 122°56'09.36"W — Elevation: 886 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | 32 | | | | | | | | | No recovery, cuttings indicate poorly cemented conglomerate, [medium dense] |
| 25 | | >72 | | | 4C | 13 | 123 | | Passing - #200 = 23.8% | | CONGLOMERATE, multi-colored clasts, highly weathered, weak to plastic, matix is weathered, plastic clay, strong rounded meta shale clasts in clayey sandstone matrix up to 2 inches, [very dense] |
| 30 | | | | | | | | | | | Harder drilling at 29 feet |
| 35 | | >66 | | | | | | | | | [Very dense] |
| | | >79 | | | | | | | | | No recovery, cuttings and returned water has greyish tinge, likely greywacke, [very dense] |

END OF BORING AT 38 FEET, Boring grouted using tremi pipe to existing ground surface

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-9 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18 San Francisco, California 94107 | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | | Sheet: 2 of 2 |

| Boring No. B-9 | Date: 11/06/12 | Drilling Agency: Gregg Drilling | Hammer Data: N/A |
|---|---|---|---|
| | Start Time: 0645 | Crew: Robert / Luis | Drill: B-53 Truck Rig |
| | End Time: 1400 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Sloping Pasture — Total Depth: 40 feet

Location: SITE 3C-2 38°24'38.92"N, 122°56'12.53"W — Elevation: 878 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | 1A | | | | | | SHALE, medium brown, completely weathered, granular to plastic |
| 5 | | | | | 2A | | | | | | SHALE, brown, highly weathered, very closely fractured, friable to plastic |
| 10 | | | | | 3A | | | | | | SHALE, bluish grey, completely to moderately weathered, very closely fractured, friable to weak, dark grey and brown, granular to plastic, sandy gravel consistency from 13.5 to 15 feet |
| 15 | | | | | 4A | | | | | | Pure SHALE, bluish black, completely to highly weathered (variable), very closely fractured, friable to weak |
| 20 | | | | | | | | | | | |

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-10 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18 San Francisco, California 94107 | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | | Sheet: 1 of 2 |

| Boring No. B-9 | Date: 11/06/12 | Drilling Agency: Gregg Drilling | Hammer Data: N/A |
|---|---|---|---|
| | Start Time: 0645 | Crew: Robert / Luis | Drill: B-53 Truck Rig |
| | End Time: 1400 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Sloping Pasture — Total Depth: 40 feet

Location: SITE 3C-2 38°24'38.92"N, 122°56'12.53"W — Elevation: 878 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Moisture Conetent % | Dry Unit Weight - pcf | Atterberg Limits | Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | | | | | | | | | | |
| | | | | | 5A | | | | | | Same |
| 25 | | | | | | | | | | | |
| | | | | | 6A | | | | | | SHALE, very dark bluish grey, highly weathered, very closely fractured, friable |
| | | | | | 7A | | | | | | META SHALE, bluish black, moderately weathered, very strong |
| 30 | | | | | | | | | | | |
| | | | | | 8A | | | | | | SHALE, dark bluish grey, highly weathered, closely fractured, friable to plastic |
| 35 | | | | | | | | | | | |
| | | | | | 9A | | | | | | SHALE, very dark bluish grey, highly to moderately weathered, very closely fractured, weak to friable, foliation visible in preferred orientation |
| 40 | | | | | | | | | | | |

END OF BORING AT 40 FEET, Boring grouted using tremi pipe to existing ground surface

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-10 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | 638 19th Street, Unit 18 San Francisco, California 94107 | Sheet: 2 of 2 |

| Boring No. B-10 | Date: 11/07/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 1100 | Crew: Robert / Luis | Drill: B-53 Truck Rig |
| | End Time: 1430 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Gently sloping ground — Total Depth: 31.5 feet

Location: SITE 3C-2 38°24'41.17"N, 122°56'11.68"W — Elevation: 881 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | 1A | | | | | | Sandstone CONGLOMERATE, pale green but mostly oxidised to yellowish red, highly to completely weathered, weak to friable to plastic, matrix with sandstone clasts, matrix varies from friable to moderately strong, fine grained sandstone |
| 5 | | | | | 2A | | | | | | No recovery, same, cuttings indicate friable to weak sandstone conglomerate |
| 10 | | | | | 3A | | | | | | No recovery, same, (logged from cuttings) |
| 15 | | | | | 4A | | | | | | No recovery, (logged from cuttings) |
| 20 | | | | | | | | | | | |

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-11 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAIMATION PROJECT OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18 San Francisco, California 94107 | |
| DATE: 01/17/2013 — APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | | Sheet: 1 of 2 |

| Boring No. B-10 | Date: 11/07/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 1100 | Crew: Robert / Luis | Drill: B-53 Truck Rig |
| | End Time: 1430 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Gently sloping ground — Total Depth: 31.5 feet

Location: SITE 3C-2 38°24'41.17"N, 122°56'11.68"W — Elevation: 881 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | | | | | | | | | | No recovery, (logged from cuttings) |
| | | | | | 5A | | | | | | |
| 25 | | 76 | | | 6B | | | | | | Sandstone CONGLOMERATE, red brown and dark brown, completely weathered, friable, [very dense] |
| | | | | | 6C | | | | TXUU | | |
| 30 | | >86 | | | 7C | | | | | | CONGLOMERATE, red brown, completely weathered, friable, composed of various rock types but mostly sandstone, [very dense] |

END OF BORING AT 31.5 FEET, Boring grouted using tremi pipe to existing ground surface

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-11 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAIMATION PROJECT OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | 638 19th Street, Unit 18 San Francisco, California 94107 | Sheet: 2 of 2 |

| Boring No. B-11 | Date: 11/07/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 0630 | Crew: Robert / Luis | Drill: B-53 Truck Rig |
| | End Time: 0845 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Grassy pasture — Total Depth: 31.5 feet

Location: SITE 3C-2 38°24'37.51"N, 122°56'12.27"W — Elevation: 866 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | | | Approximately 6 inches topsoil/colluvium |
| 5 | | 50/3" | | | 1A | | | | | | SERPENTINITE, generally dark green, highly weathered, friable and plastic, dry, [very dense], highly variable consistency, color and engineering properties, colors include greys, greens, and brown, horizons of clay, sandy, gravelly, and rock material |
| 10 | | 23 | | | 2B<br>2C | | | LL 39<br>PI 21 | TXUU | | Same, [medium dense] |
| 15 | | 50/3" | | | 3A | | | | | | Same, dark green-grey, [very dense] |
| 20 | | | | | | | | | | | |

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-12 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18<br>San Francisco, California 94107 | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | | Sheet: 1 of 2 |

| Boring No. B-11 | Date: 11/07/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 0630 | Crew: Robert / Luis | Drill: B-53 Truck Rig |
| | End Time: 0845 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Grassy pasture — Total Depth: 31.5 feet

Location: SITE 3C-2 38°24'37.51"N, 122°56'12.27"W — Elevation: 866 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | 42 | | | 4B<br>4C | | | LL 42<br>PI 24 | TXCU w/ PP c'=0.198ksf Φ'=22.3° | | SERPENTINITE, greys/greens/browns, variable material strong rock to fat clay, [dense] |
| 25 | | 50/6" | | | 5A | | | | | | Same, [very dense] |
| 30 | | 63 | | | 6B<br>6C | | | | | | Same, [dense] |

END OF BORING AT 31.5 FEET, Boring grouted using tremi pipe to existing ground surface

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-12 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18 San Francisco, California 94107 | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | | Sheet: 2 of 2 |

| Boring No. B-12 | Date: 11/17/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| | Start Time: 0810 | Crew: Jason / Daniel | Drill: MARL M5 Track Rig (Rhino Rig) |
| | End Time: 0930 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Fern covered forest floor — Total Depth: 35 feet

Location: SITE 3C-2 38°24'40.79"N, 122°56'12.69"W — Elevation: 876 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | | | Organic sandy topsoil, brown and dark reddish brown |
| | | 32 | | | 1A | | | | | | Sandstone CONGLOMERATE, light bluish grey, highly to moderately weathered, friable, [medium dense] |
| | | | | | 1B | | | | | | |
| 5 | | | | | 1C | | | | TXCU w/ PP c'=0.073ksf Φ'=33.1° | | |
| | | 31 | | | 2B | | | | | | Sandstone CONGLOMERATE, reddish yellow clasts, light bluish grey matrix, angular sandstone clasts, 0.5 to 2 inches, clay layers highly weathered, friable, [medium dense] |
| | | | | | 2C | | | | | | |
| 10 | | | | | | | | | | | |
| | | 24 | | | 3A | | | LL 41 | | | Sandstone CONGLOMERATE, multi-colored, highly weathered, friable to plastic, [medium dense] |
| | | | | | 3B | | | PI 13 | TXUU | | |
| | | | | | 3C | | | | | | |
| 15 | | | | | | | | | | | |
| | | 18 | | | 4A | | | | TXUU | | Sandstone CONGLOMERATE, orange clasts, pale green matrix, highly weathered, matrix is plastic and friable, clasts are coarse grained, mostly sandstone, [moderately dense] |
| | | | | | 4B | | | LL 40 | | | |
| | | | | | 4C | | | PI 14 | | | |
| 20 | | | | | | | | | | | |

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-13 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | 638 19th Street, Unit 18 San Francisco, California 94107 | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | | Sheet: 1 of 2 |

| Boring No. | Date: 11/17/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| B-12 | Start Time: 0810 | Crew: Jason / Daniel | Drill: MARL M5 Track Rig (Rhino Rig) |
| | End Time: 0930 | Geologist: Alastair Johns | Type of Bit: Tricone |

Surface Conditions: Fern covered forest floor — Total Depth: 35 feet

Location: SITE 3C-2 38°24'40.79"N, 122°56'12.69"W — Elevation: 876 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | 53 | | | 5A, 5B, 5C | | | | | | GREYWACKE, bluish grey, highly to moderately weathered, friable, generally poorly sorted, layers of fine grained sandstone, [dense] |
| 25 | | 46 | | | 6A, 6B, 6C | 7 | 128 | | | | GREYWACKE, bluish grey, moderately weathered, friable, fine grained sandstone with shale horizons, [dense] |
| 30 | | 50/6" | | | 7A, 7B | | | | | | GREYWACKE, bluish grey, moderately weathered, weak, fine grained sandstone, [very dense] |
| 35 | | | | | | | | | | | |

END OF BORING AT 35 FEET, Boring grouted using tremi pipe to existing ground surface

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-13 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | 638 19th Street, Unit 18 San Francisco, California 94107 | Sheet: 2 of 2 |

| Boring No. | Date: 11/17/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| B-13 | Start Time: 1000 | Crew: Jason / Daniel / Jeremy | Drill: MARL M5 Track Rig (Rhino Rig) |
| | End Time: 1115 | Geologist: Alastair Johns | Type of Bit: N/A |

Surface Conditions: Grassy pasture — Total Depth: 36.5 feet

Location: SITE 3C-2 38°24'37.84"N, 122°56'12.53"W — Elevation: 865 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | | | |
| 5 | | 50/6" | | | | | | | | | |
| | | | | | 1C | | | | | | SERPENTINITE, light green-grey, highly weathered, friable, [very dense] |
| | | | | | | | | | | | Color change to grey at 10 feet |
| 10 | | 56 | | | 2B | | | | | | |
| | | | | | 2C | 7 | 121 | | | | SERPENTINITE, bluish grey, highly weathered, plastic, [medium dense] |
| 15 | | 66 | | | 3A | | | | | | |
| | | | | | 3B | | | | | | |
| | | | | | 3C | | | | | | SERPENTINITE, same, [dense] |
| 20 | | | | | | | | | | | |

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-14 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJCET OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | 638 19"' Street, Unit 18 San Francisco, California 94107 | Sheet: 1 of 2 |

| Boring No. | Date: 11/17/12 | Drilling Agency: Gregg Drilling | Hammer Data: 140 lbs, 30 inch drop, Auto Hammer |
|---|---|---|---|
| B-13 | Start Time: 1000 | Crew: Jason / Daniel / Jeremy | Drill: MARL M5 Track Rig (Rhino Rig) |
| | End Time: 1115 | Geologist: Alastair Johns | Type of Bit: N/A |

Surface Conditions: Grassy pasture — Total Depth: 36.5 feet

Location: SITE 3C-2 38°24'37.84"N, 122°56'12.53"W — Elevation: 865 feet amsl

| Depth | Recovery Method | Field Tests: Blows / ft Counts | Field Tests: Torvane - tsf | Field Tests: Pocket - tsf Penetrometer | Laboratory Tests: Sample No. | Laboratory Tests: Moisture Conetent % | Laboratory Tests: Dry Unit Weight - pcf | Laboratory Tests: Atterberg Limits | Laboratory Tests: Other Tests | Graphic Symbol | USCS Description |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 20 | | 50/6" | | | 4A | 11 | 115 | | | | SERPENTINITE, same, [very dense] |
| 25 | | 50/6" | | | 5B<br>5C | | | | | | SERPENTINITE, same, [very dense] |
| 30 | | 50/4" | | | 6C | 10 | 111 | | | | SERPENTINITE, same, [very dense] |
| 35 | | 54 | | | 7C | | | | | | |

END OF BORING AT 36.5 FEET, Boring grouted using tremi pipe to existing ground surface

| | | | |
|---|---|---|---|
| COMPILED BY: N. Scheemaker | **BORING LOG** | M2 CONSULTANTS inc | Plate No. |
| REVISED BY: N. Scheemaker | | | A-14 |
| CHECKED BY: A. Johns | OCSD WASTEWATER STORAGE AND RECLAMATION PROJCET<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/17/2013 / APPROVED BY: Y. Prashar | PROJECT NO: 01202 / Geo 2 | 638 19"' Street, Unit 18<br>San Francisco, California 94107 | Sheet: 2 of 2 |

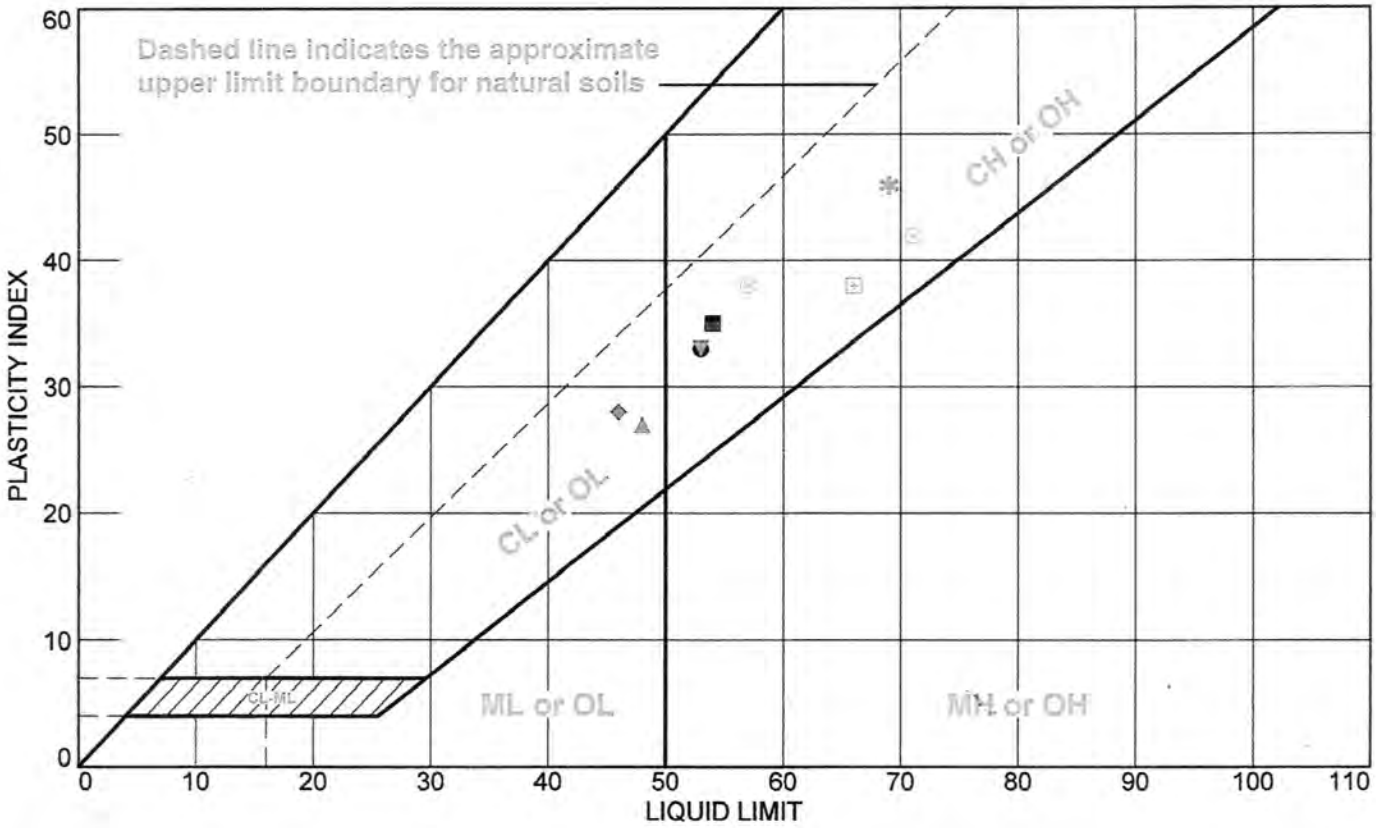
# APPENDIX B

# LABORATORY TESTS

# SITE 3A

# PLATES B-1 THROUGH B-12

# LIQUID AND PLASTIC LIMITS TEST REPORT




## SOIL DATA

| | SOURCE | SAMPLE NO. | DEPTH | NATURAL WATER CONTENT (%) | PLASTIC LIMIT (%) | LIQUID LIMIT (%) | PLASTICITY INDEX (%) | LIQUIDITY INDEX | USCS |
|---|---|---|---|---|---|---|---|---|---|
| ● | | B-1 1A | 1-3.5' | | 20 | 53 | 33 | | CH |
| ■ | | B-1 3A | 5-7.5' | | 19 | 54 | 35 | | SC |
| ▲ | | B-1 5B | 10.5' | | 21 | 48 | 27 | | SC |
| ◆ | | B-2 1A | 1.5-4' | | 18 | 46 | 28 | | CL |
| ▼ | | B-2 3A | 5.5-8' | | 20 | 53 | 33 | | CH |
| * | | B-2 4C | 9.0' | | 23 | 69 | 46 | | SC |
| ⊕ | | B-3 1C | 4.0' | | 29 | 71 | 42 | | CH |
| ⊞ | | B-3 2B | 5.5' | | 28 | 66 | 38 | | CL |
| ⊗ | | B-4 5B | 10.5' | | 19 | 57 | 38 | | CH |

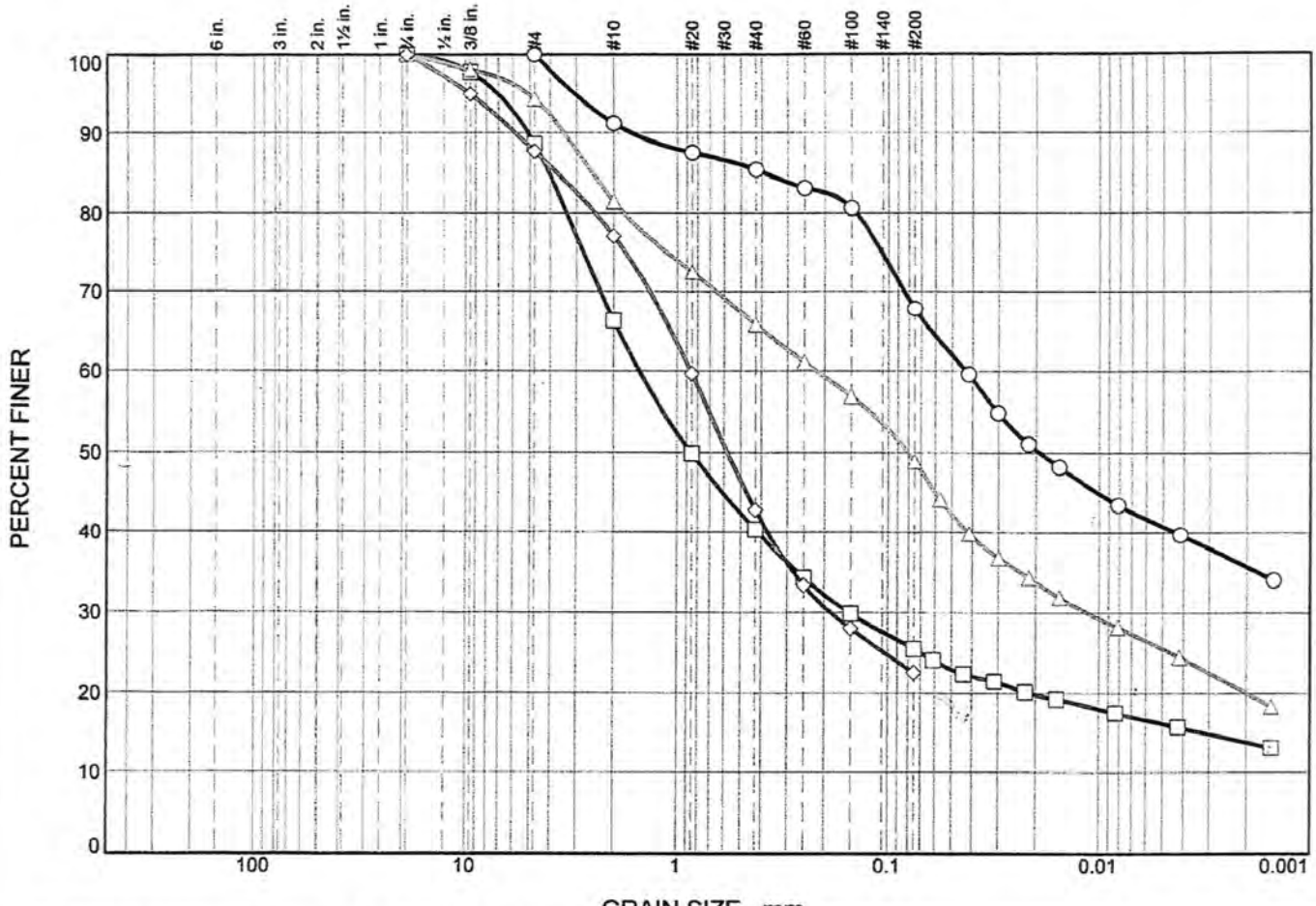
**Soil Mechanics Lab**

**Oakland, California**

**Client:** M2 Consultants,Inc.
**Project:** OCSD Waste Water Storage and Reclamation Project.

**Project No.:** 1202 **Plate** B-1

# Particle Size Distribution Report




| | % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|---|
| | | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| ○ | 0.0 | 0.0 | 0.0 | 8.8 | 5.8 | 17.6 | 27.1 | 40.7 |
| □ | 0.0 | 0.0 | 11.4 | 22.3 | 25.9 | 14.9 | 9.3 | 16.2 |
| △ | 0.0 | 0.0 | 5.6 | 12.9 | 15.7 | 16.9 | 23.4 | 25.5 |
| ◇ | 0.0 | 0.0 | 12.3 | 10.6 | 34.3 | 20.2 | 22.6 | |
| | | | | | | | | |

## SOIL DATA

| SYMBOL | SOURCE | SAMPLE NO. | DEPTH (ft.) | Material Description | USCS |
|---|---|---|---|---|---|
| ○ | B-1 | 1A | 1-3.5' | Black FAT CLAY with sand. | CH |
| □ | B-1 | | | See remarks. | SC |
| △ | B-1 | 5B | 10.5' | ark grey clayey SAND. | SC |
| ◇ | B-1 | 5C | 11' | Dark grey clayey SAND. | SC |
| | | | | | |

**Soil Mechanics Lab**

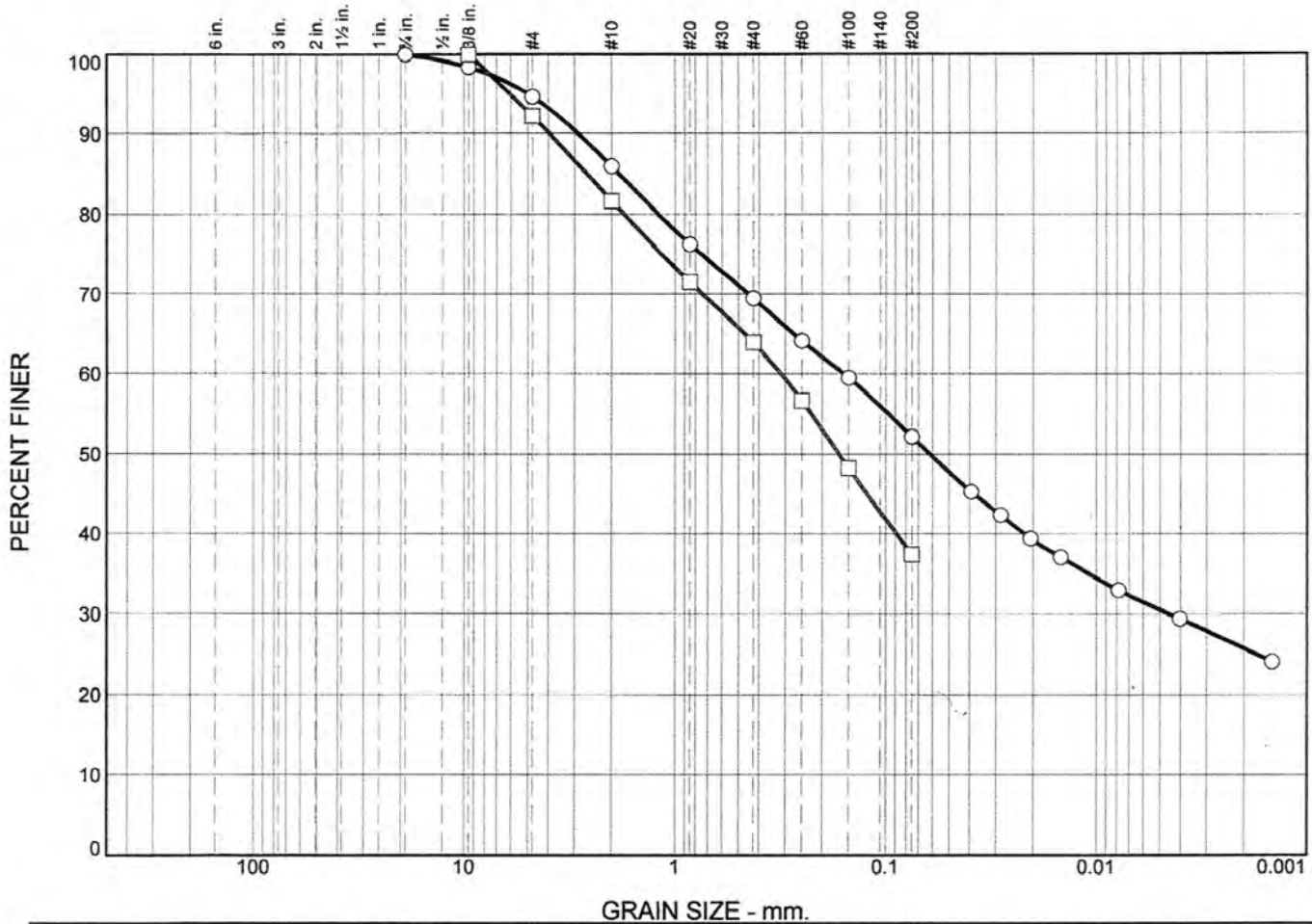
**Oakland, California**

**Client:** M2 Consultants,Inc.

**Project:** OCSD Waste Water Storage and Reclamation Project.

**Project No.:** 1202 **Plate** B-2

# Particle Size Distribution Report




| | % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|---|
| | | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| ○ | 0.0 | 0.0 | 5.4 | 8.7 | 16.4 | 17.3 | 21.7 | 30.5 |
| □ | 0.0 | 0.0 | 7.8 | 10.5 | 17.8 | 26.5 | 37.4 | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

## SOIL DATA

| SYMBOL | SOURCE | SAMPLE NO. | DEPTH (ft.) | Material Description | USCS |
|---|---|---|---|---|---|
| ○ | B-2 | 1A | 1.5-4' | Brownish black sandy CLAY. | CL |
| □ | B-2 | 4C | 9.0 | Dark brown clayey SAND. | SC |
| | | | | | |
| | | | | | |
| | | | | | |

**Soil Mechanics Lab**

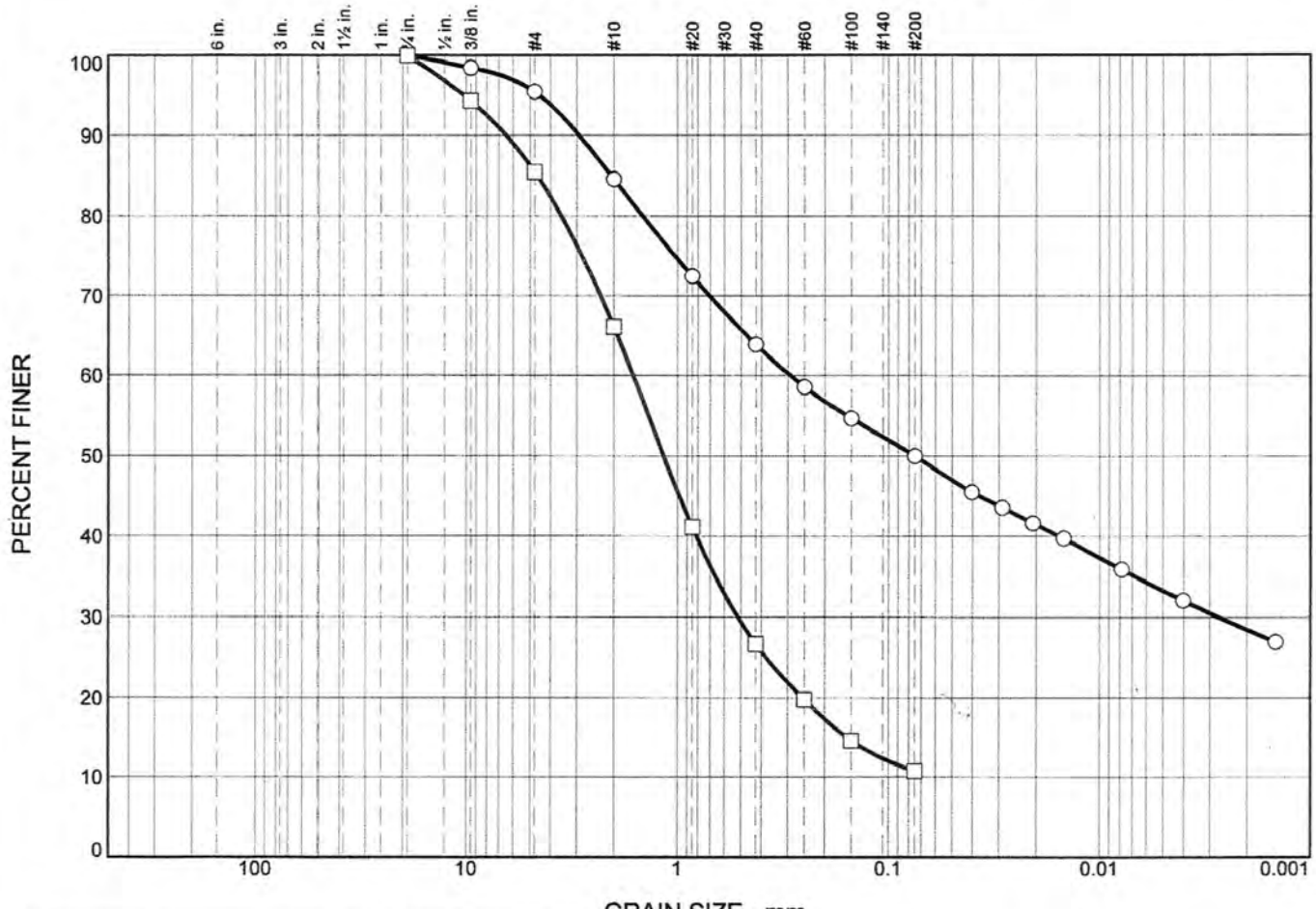
**Oakland, California**

**Client:** M2 Consultants,Inc.

**Project:** OCSD Waste Water Storage and Reclamation Project.

**Project No.:** 1202 **Plate** B-3

# Particle Size Distribution Report




| | % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|---|
| | | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| ○ | 0.0 | 0.0 | 4.5 | 11.0 | 20.6 | 13.9 | 16.7 | 33.3 |
| □ | 0.0 | 0.0 | 14.6 | 19.3 | 39.4 | 16.0 | 10.7 | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

## SOIL DATA

| SYMBOL | SOURCE | SAMPLE NO. | DEPTH (ft.) | Material Description | USCS |
|---|---|---|---|---|---|
| ○ | B-3 | 2B | 5.5' | Very dark grey sany CLAY. | CL |
| □ | B-3 | 3C | 8.0' | Brownish Blacl clayey SAND with gravel. | SC |
| | | | | | |
| | | | | | |
| | | | | | |

**Soil Mechanics Lab**

**Oakland, California**

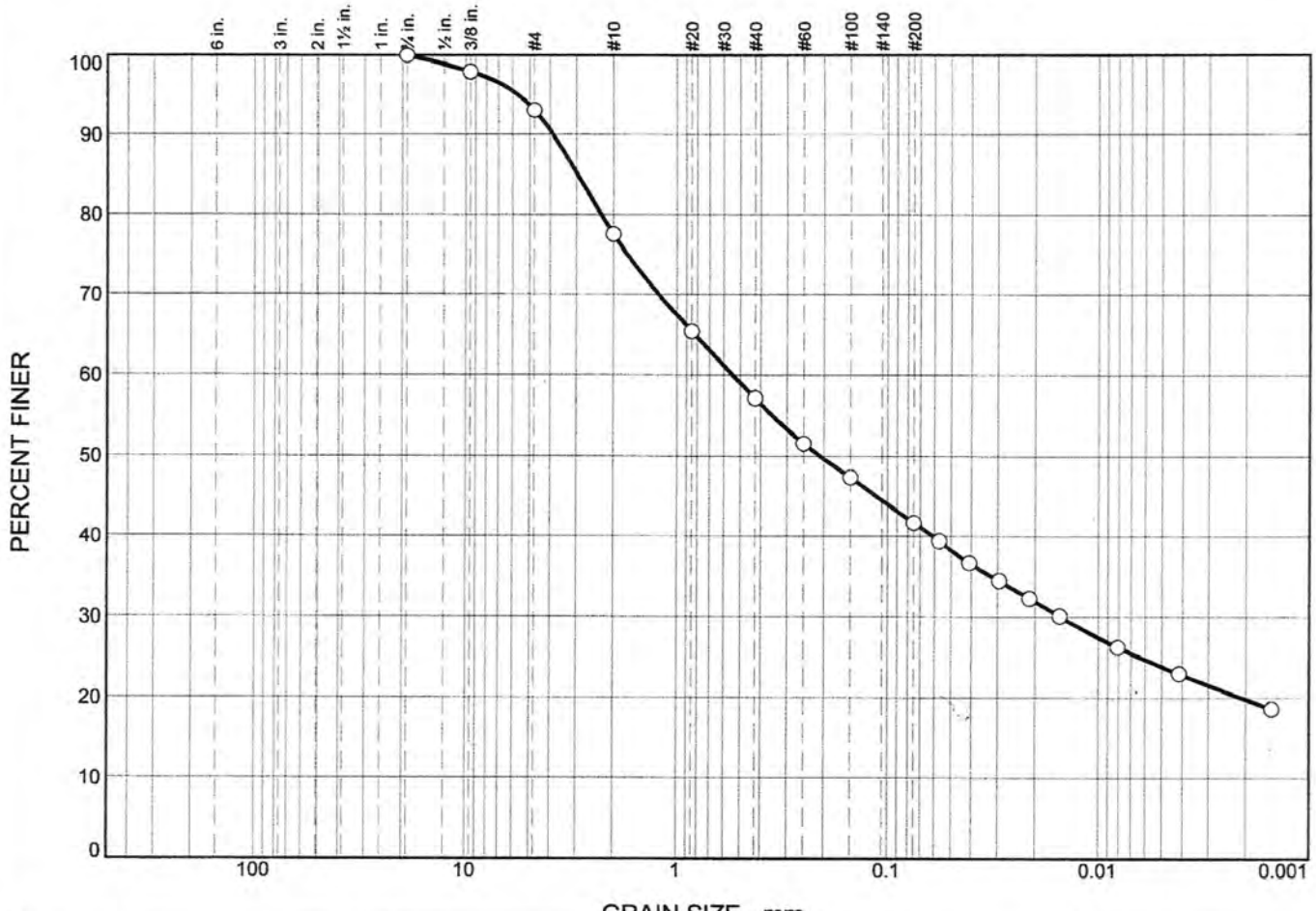
**Client:** M2 Consultants,Inc.

**Project:** OCSD Waste Water Storage and Reclamation Project.

**Project No.:** 1202

**Plate** B-4

# Particle Size Distribution Report




| | % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|---|
| | | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| ○ | 0.0 | 0.0 | 6.9 | 15.6 | 20.4 | 15.4 | 17.8 | 23.9 |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

## SOIL DATA

| SYMBOL | SOURCE | SAMPLE NO. | DEPTH (ft.) | Material Description | USCS |
|---|---|---|---|---|---|
| ○ | B-4 | 5B | 10.5' | Dark brown clayey SAND. | SC |
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |

**Soil Mechanics Lab**

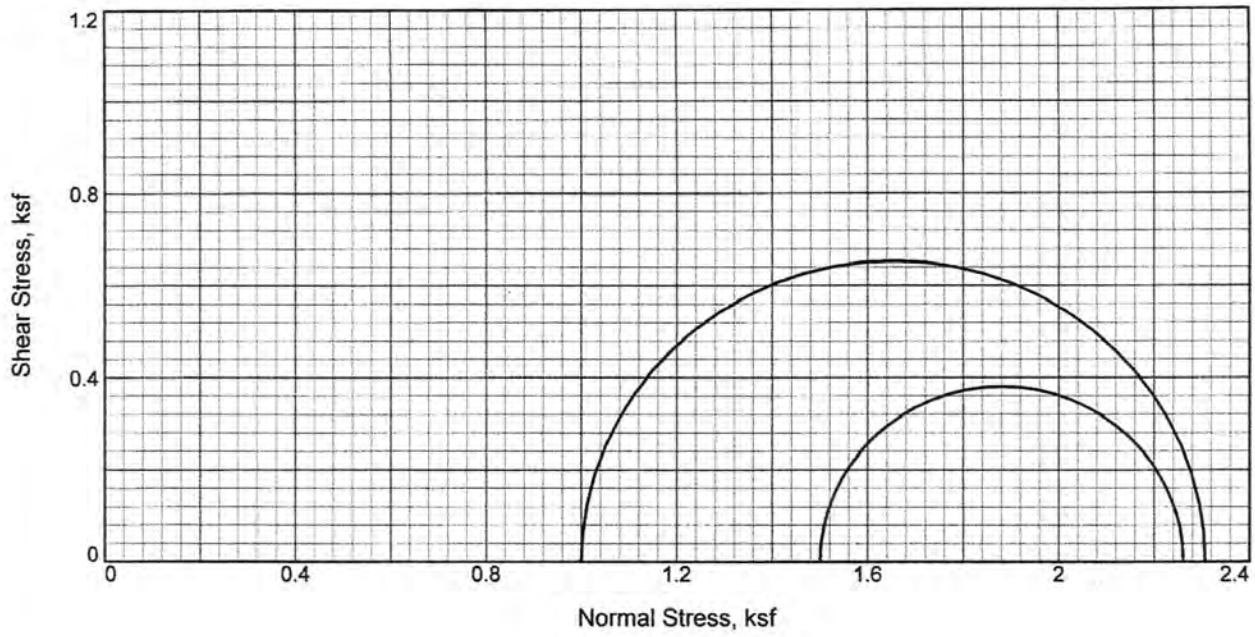
**Oakland, California**

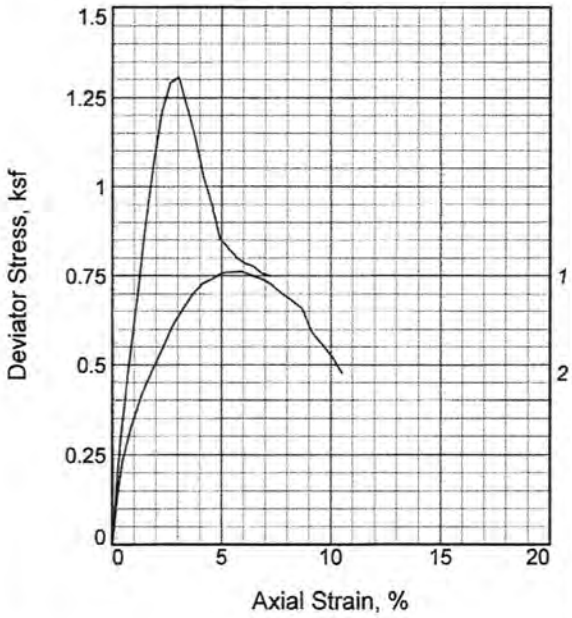
**Client:** M2 Consultants,Inc.

**Project:** OCSD Waste Water Storage and Reclamation Project.

**Project No.:** 1202

**Plate** B-5




| Sample No. | | 1 | 2 |
|---|---|---|---|
| Initial | Water Content, % | 30.3 | 30.9 |
| | Dry Density, pcf | 91.6 | 90.1 |
| | Saturation, % | 97.4 | 95.8 |
| | Void Ratio | 0.8406 | 0.8702 |
| | Diameter, in. | 2.875 | 2.416 |
| | Height, in. | 5.280 | 4.380 |
| At Test | Water Content, % | 30.3 | 30.9 |
| | Dry Density, pcf | 91.6 | 90.1 |
| | Saturation, % | 97.4 | 95.8 |
| | Void Ratio | 0.8406 | 0.8702 |
| | Diameter, in. | 2.875 | 2.416 |
| | Height, in. | 5.280 | 4.380 |
| Strain rate, in./min. | | 0.075 | 0.075 |
| Back Pressure, ksf | | 0.00 | 0.00 |
| Cell Pressure, ksf | | 1.00 | 1.50 |
| Fail. Stress, ksf | | 1.31 | 0.76 |
| Strain, % | | 3.0 | 5.9 |
| Ult. Stress, ksf | | | |
| Strain, % | | | |
| $\sigma_1$ Failure, ksf | | 2.31 | 2.26 |
| $\sigma_3$ Failure, ksf | | 1.00 | 1.50 |

**Type of Test:**
Unconsolidated Undrained
**Sample Type:** Shelby
**Description:** See remarks.

**LL=** 54 **PL=** 19 **PI=** 35
**Assumed Specific Gravity=** 2.70
**Remarks:** #1/B-1,3A @ 5':Dark grey clayey SAND(SC)
1,5B @ 10.5':Dark grey clayey SAND(SC)

**Client:** M2 Consultants,Inc.

**Project:** OCSD Waste Water Storage and Reclamation Project.

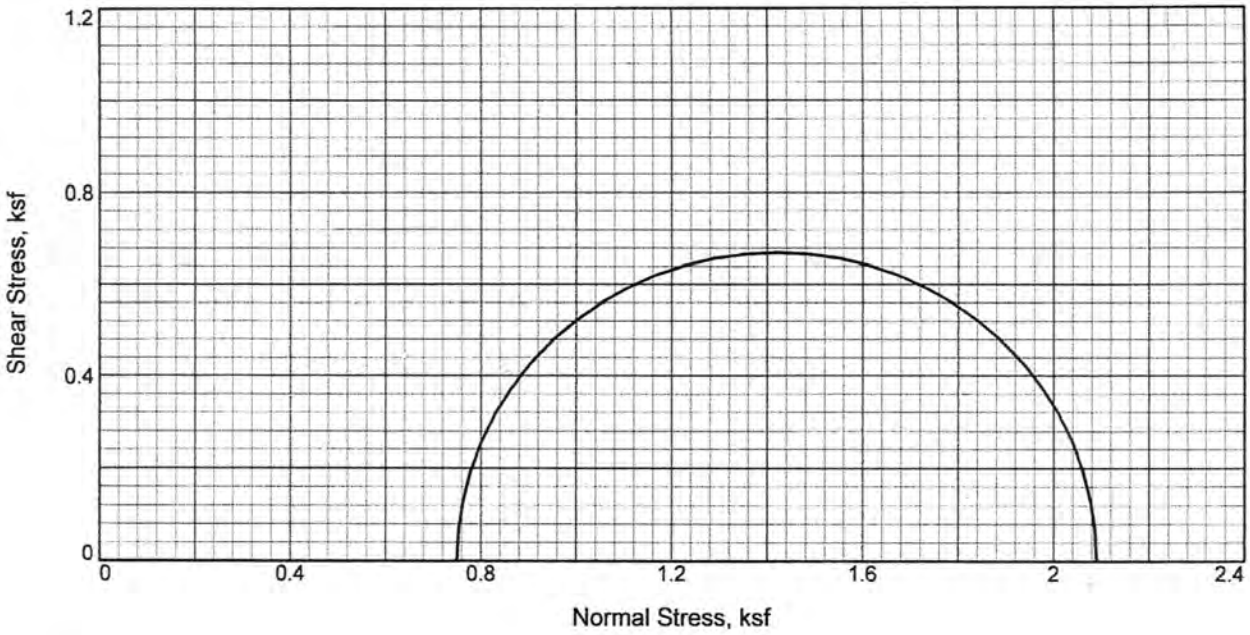
**Source of Sample:** B-1

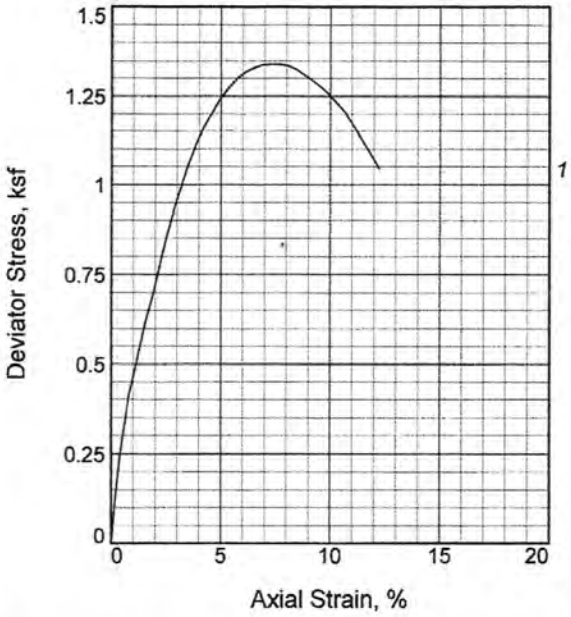
**Proj. No.:** 1202 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT

# Soil Mechanics Lab

**Plate** B-6




| | | |
|---|---|---|
| Sample No. | | 1 |
| Initial | Water Content, % | 27.7 |
| | Dry Density, pcf | 95.5 |
| | Saturation, % | 98.0 |
| | Void Ratio | 0.7642 |
| | Diameter, in. | 2.875 |
| | Height, in. | 5.230 |
| At Test | Water Content, % | 27.7 |
| | Dry Density, pcf | 95.5 |
| | Saturation, % | 98.0 |
| | Void Ratio | 0.7642 |
| | Diameter, in. | 2.875 |
| | Height, in. | 5.230 |
| Strain rate, in./min. | | 0.750 |
| Back Pressure, ksf | | 0.00 |
| Cell Pressure, ksf | | 0.75 |
| Fail. Stress, ksf | | 1.34 |
| Strain, % | | 7.6 |
| Ult. Stress, ksf | | |
| Strain, % | | |
| $\sigma_1$ Failure, ksf | | 2.09 |
| $\sigma_3$ Failure, ksf | | 0.75 |

**Type of Test:**
Unconsolidated Undrained
**Sample Type:** Shelby
**Description:** Brownish black sandy CLAY.

**LL=** 46 **PL=** 18 **PI=** 28
**Assumed Specific Gravity=** 2.70
**Remarks:**

**Client:** M2 Consultants,Inc.

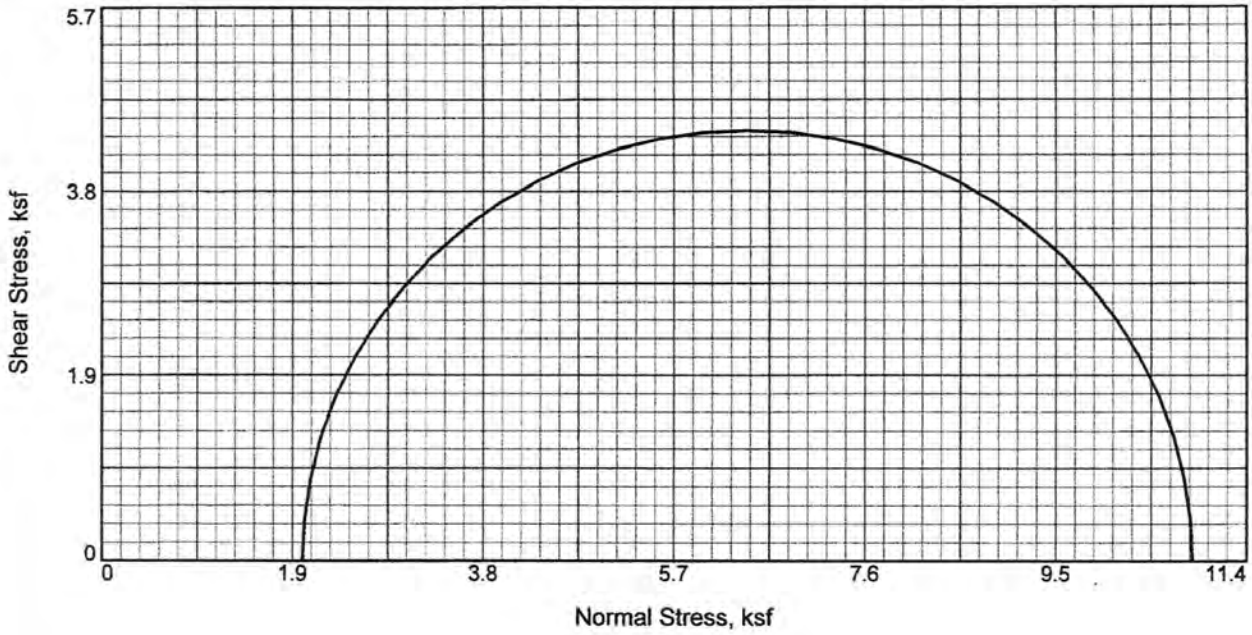
**Project:** OCSD Waste Water Storage and Reclamation Project.

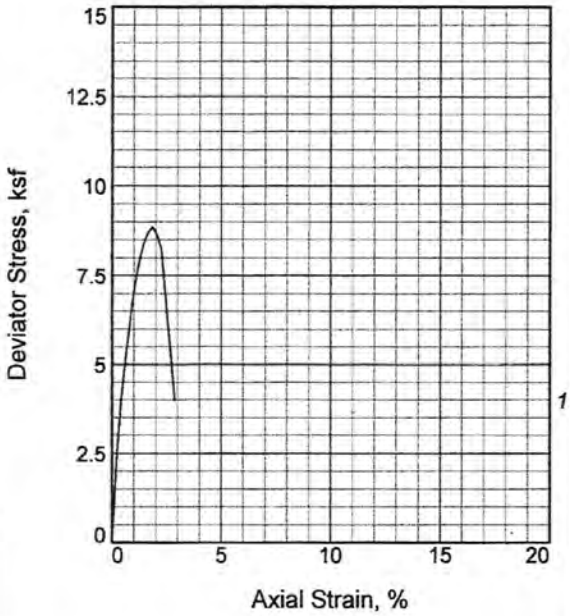
**Location:** B-2
**Sample Number:** 1A **Depth:** 1.5-4'
**Proj. No.:** 1202 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT

# Soil Mechanics Lab

**Plate** B-7




| | | |
|---|---|---|
| Sample No. | | 1 |
| Initial | Water Content, % | 9.4 |
| | Dry Density, pcf | 128.6 |
| | Saturation, % | 81.7 |
| | Void Ratio | 0.3112 |
| | Diameter, in. | 2.420 |
| | Height, in. | 4.880 |
| At Test | Water Content, % | 9.4 |
| | Dry Density, pcf | 128.6 |
| | Saturation, % | 81.7 |
| | Void Ratio | 0.3112 |
| | Diameter, in. | 2.420 |
| | Height, in. | 4.880 |
| Strain rate, in./min. | | 0.075 |
| Back Pressure, ksf | | 0.0 |
| Cell Pressure, ksf | | 2.0 |
| Fail. Stress, ksf | | 8.9 |
| Strain, % | | 1.8 |
| Ult. Stress, ksf | | |
| Strain, % | | |
| $\sigma_1$ Failure, ksf | | 10.9 |
| $\sigma_3$ Failure, ksf | | 2.0 |

**Type of Test:**
Unconsolidated Undrained

**Sample Type:** Core

**Description:** Light to med.green weathered Serpentine Bedrock with meta-shale clasts.Friable. Fines fat clay(CH) like.

**Assumed Specific Gravity=** 2.70

**Remarks:**

**Client:** M2 Consultants,Inc.

**Project:** OCSD Waste Water Storage and Reclamation Project.

**Location:** B-3

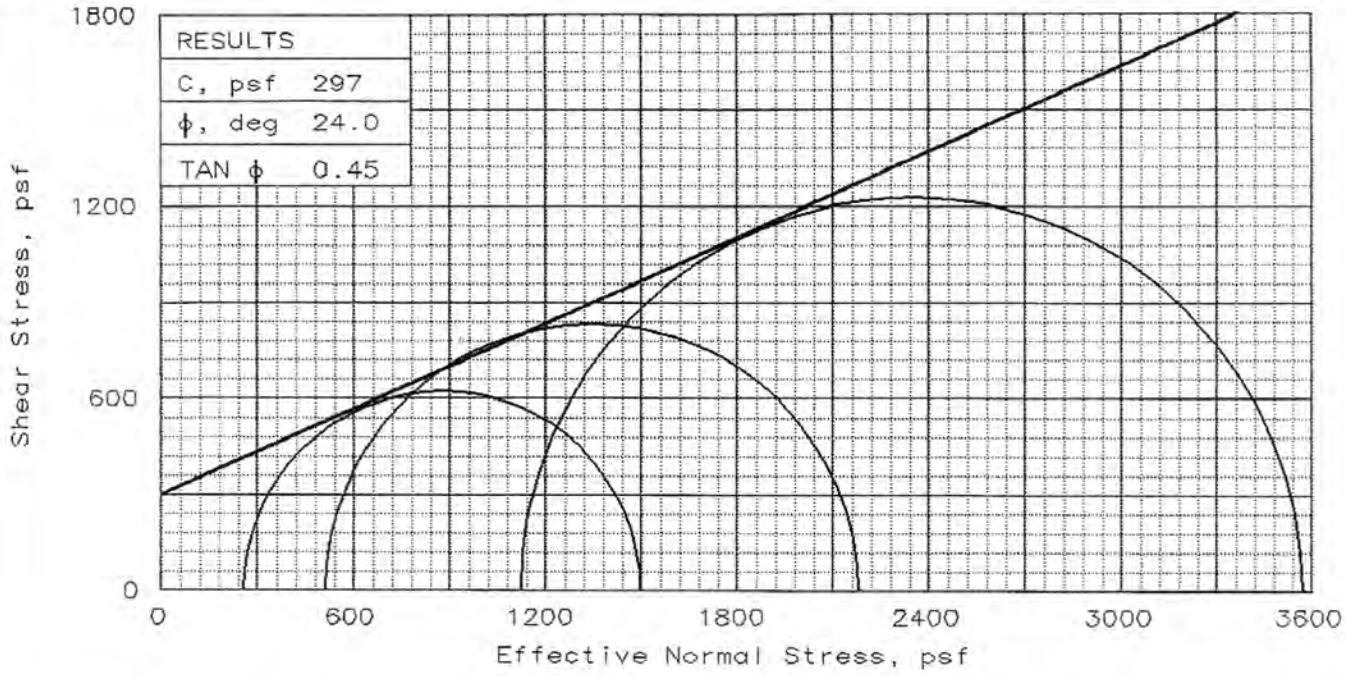
**Sample Number:** 6A **Depth:** 15-17'

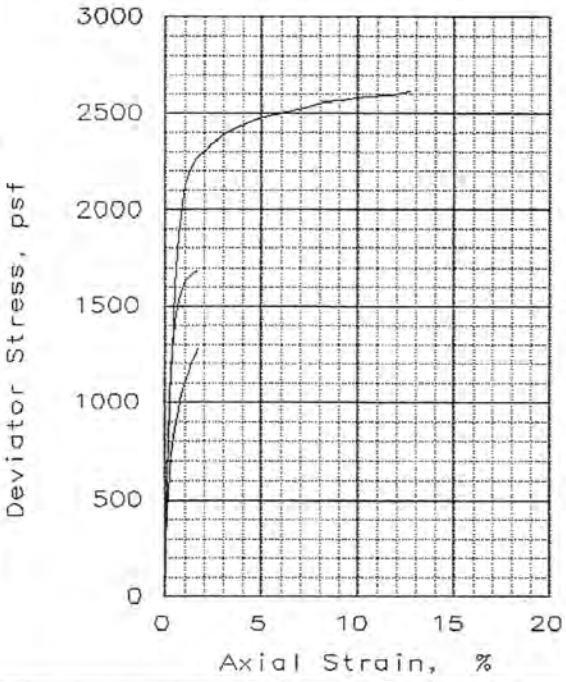
**Proj. No.:** 1202 **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT

Soil Mechanics Lab

**Plate** B-8




| | Specimen No.: | 1 | 2 | 3 |
|---|---|---|---|---|
| INITIAL | WATER CONTENT, % | 26.6 | 26.6 | 26.6 |
| | DRY DENSITY, pcf | 97.9 | 97.9 | 97.9 |
| | SATURATION, % | 99.6 | 99.6 | 99.6 |
| | VOID RATIO | 0.721 | 0.721 | 0.721 |
| | DIAMETER, in | 2.88 | 2.88 | 2.88 |
| | HEIGHT, in | 5.71 | 5.71 | 5.71 |
| AT TEST | WATER CONTENT, % | 26.6 | 26.6 | 26.6 |
| | DRY DENSITY, pcf | 98.0 | 98.5 | 99.6 |
| | SATURATION, % | 100.0 | 101.0 | 104.0 |
| | VOID RATIO | 0.720 | 0.712 | 0.692 |
| | DIAMETER, in | 2.87 | 2.89 | 2.90 |
| | HEIGHT, in | 5.71 | 5.61 | 5.52 |
| Strain rate, in/min | | 0.0030 | 0.0030 | 0.0030 |
| EFF CELL PRESSURE, psf | | 500 | 1000 | 2000 |
| FAIL. STRESS, psf | | 1242 | 1667 | 2449 |
| EXCESS PORE PR., psf | | 241 | 486 | 874 |
| STRAIN, % | | 1.6 | 1.5 | 4.2 |
| ULT. STRESS, psf | | | | |
| EXCESS PORE PR., psf | | | | |
| STRAIN, % | | | | |
| $\bar{\sigma}_1$ FAILURE, psf | | 1501 | 2182 | 3575 |
| $\bar{\sigma}_3$ FAILURE, psf | | 259 | 514 | 1126 |

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: Shelby
DESCRIPTION: Black FAT CLAY
with sand(CH)

SPECIFIC GRAVITY= 2.7
REMARKS:

CLIENT: M2 Consultants,Inc.

PROJECT: OCSD

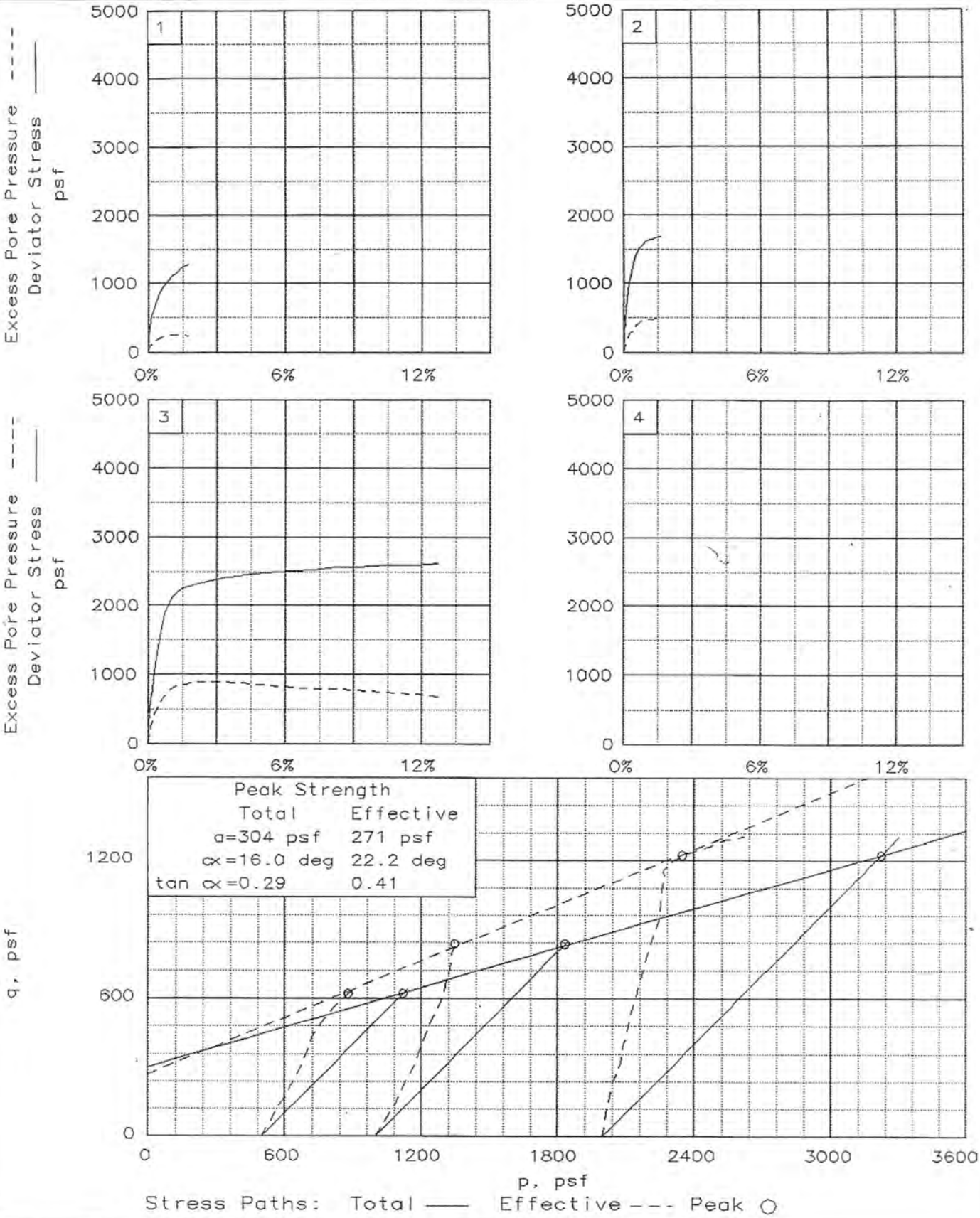
SAMPLE LOCATION: B-1 1A 1-3.5'

PROJ. NO.: 1202 DATE: 3-12-12

TRIAXIAL SHEAR TEST REPORT

**Soil Mechanics Lab**

Plate No: B-9

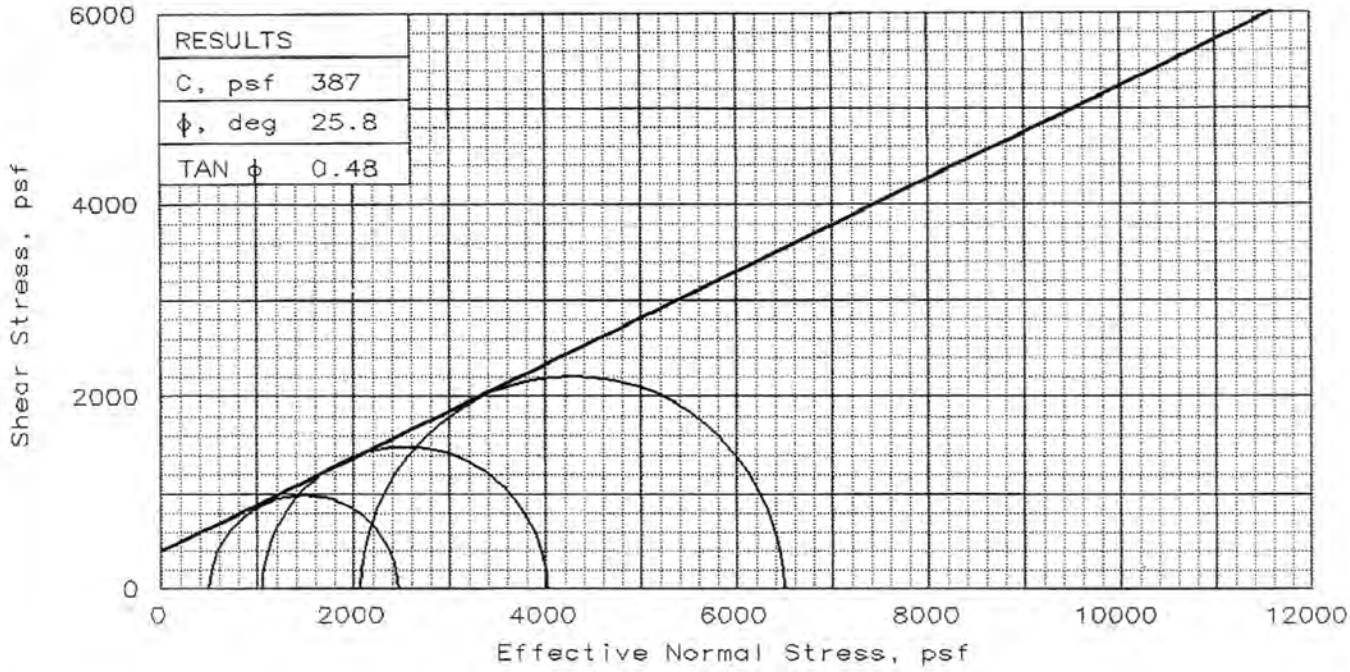


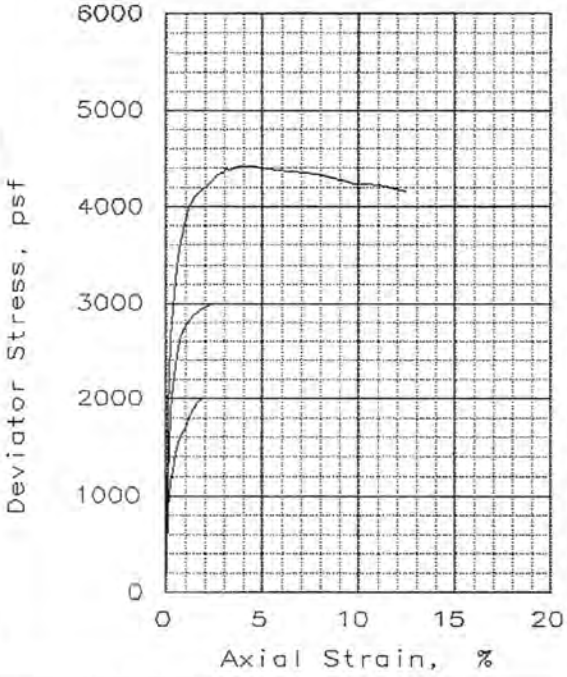

Client: M2 Consultants,Inc.

Project: OCSD

Location: B-1 1A 1-3.5'

File: Project No.: 1202 Plate No: B-9A




| Specimen No.: | | 1 | 2 | 3 |
|---|---|---|---|---|
| INITIAL | WATER CONTENT, % | 36.6 | 36.6 | 36.6 |
| | DRY DENSITY, pcf | 85.8 | 85.8 | 85.8 |
| | SATURATION, % | 102.5 | 102.5 | 102.5 |
| | VOID RATIO | 0.964 | 0.964 | 0.964 |
| | DIAMETER, in | 2.88 | 2.88 | 2.88 |
| | HEIGHT, in | 5.48 | 5.48 | 5.48 |
| AT TEST | WATER CONTENT, % | 34.8 | 34.8 | 34.8 |
| | DRY DENSITY, pcf | 86.9 | 88.1 | 89.9 |
| | SATURATION, % | 100.1 | 102.9 | 107.6 |
| | VOID RATIO | 0.939 | 0.914 | 0.874 |
| | DIAMETER, in | 2.86 | 2.86 | 2.87 |
| | HEIGHT, in | 5.48 | 5.38 | 5.26 |
| Strain rate, in/min | | 0.0030 | 0.0030 | 0.0030 |
| EFF CELL PRESSURE, psf | | 1000 | 2000 | 4000 |
| FAIL. STRESS, psf | | 1966 | 2962 | 4413 |
| EXCESS PORE PR., psf | | 490 | 936 | 1915 |
| STRAIN, % | | 1.6 | 2.0 | 4.0 |
| ULT. STRESS, psf | | | | |
| EXCESS PORE PR., psf | | | | |
| STRAIN, % | | | | |
| $\bar{\sigma}_1$ FAILURE, psf | | 2476 | 4026 | 6498 |
| $\bar{\sigma}_3$ FAILURE, psf | | 510 | 1064 | 2085 |

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: Shelby
DESCRIPTION: Stiff,dark brown
FAT CLAY(CH)w/c-sand.

SPECIFIC GRAVITY= 2.7
REMARKS:

CLIENT: M2 Consultants,Inc.

PROJECT: OCSD

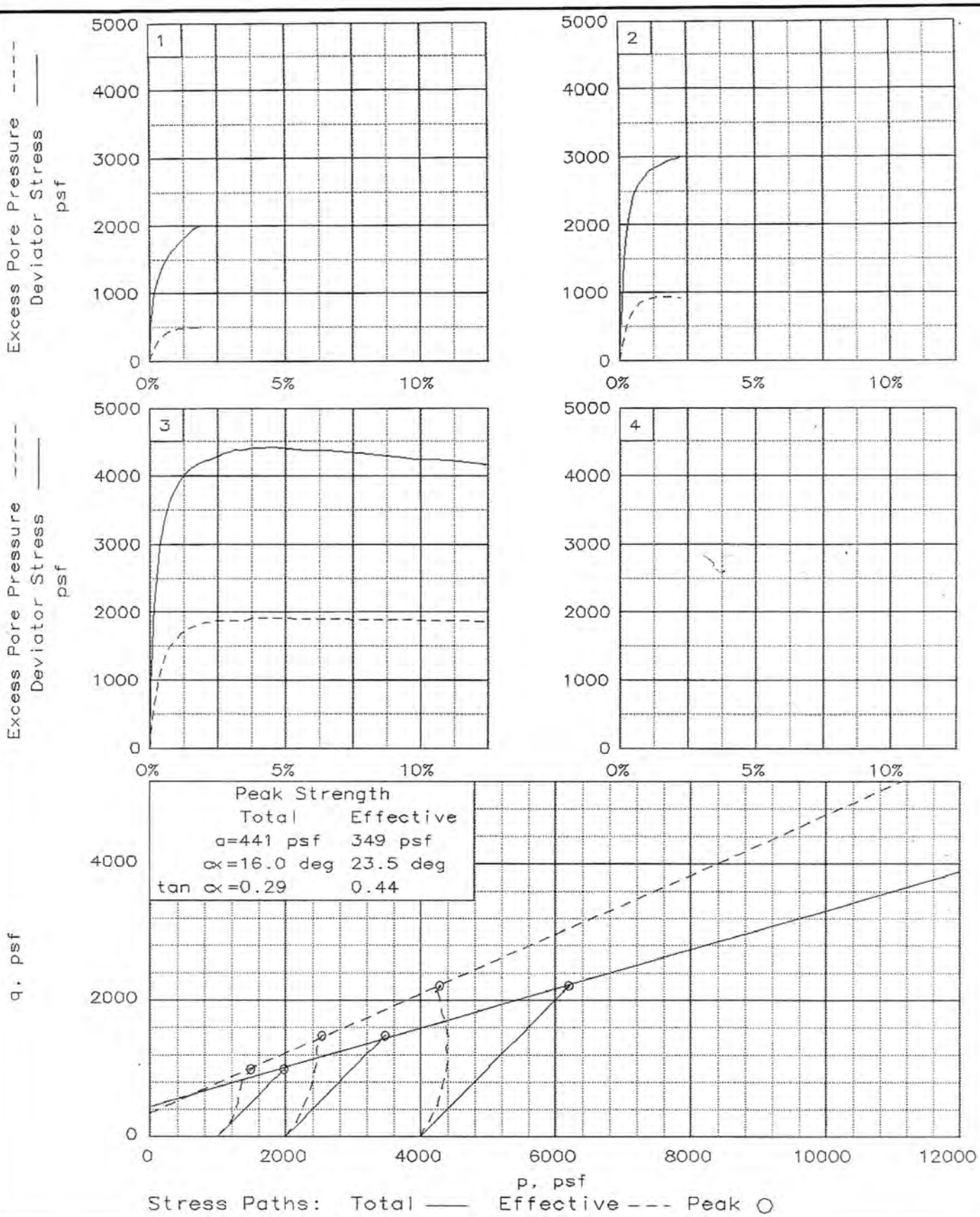
SAMPLE LOCATION: B-2 3A 5.5-8'

PROJ. NO.: 1202 DATE: 3-12-12

TRIAXIAL SHEAR TEST REPORT

**Soil Mechanics Lab**

Plate No: B-10

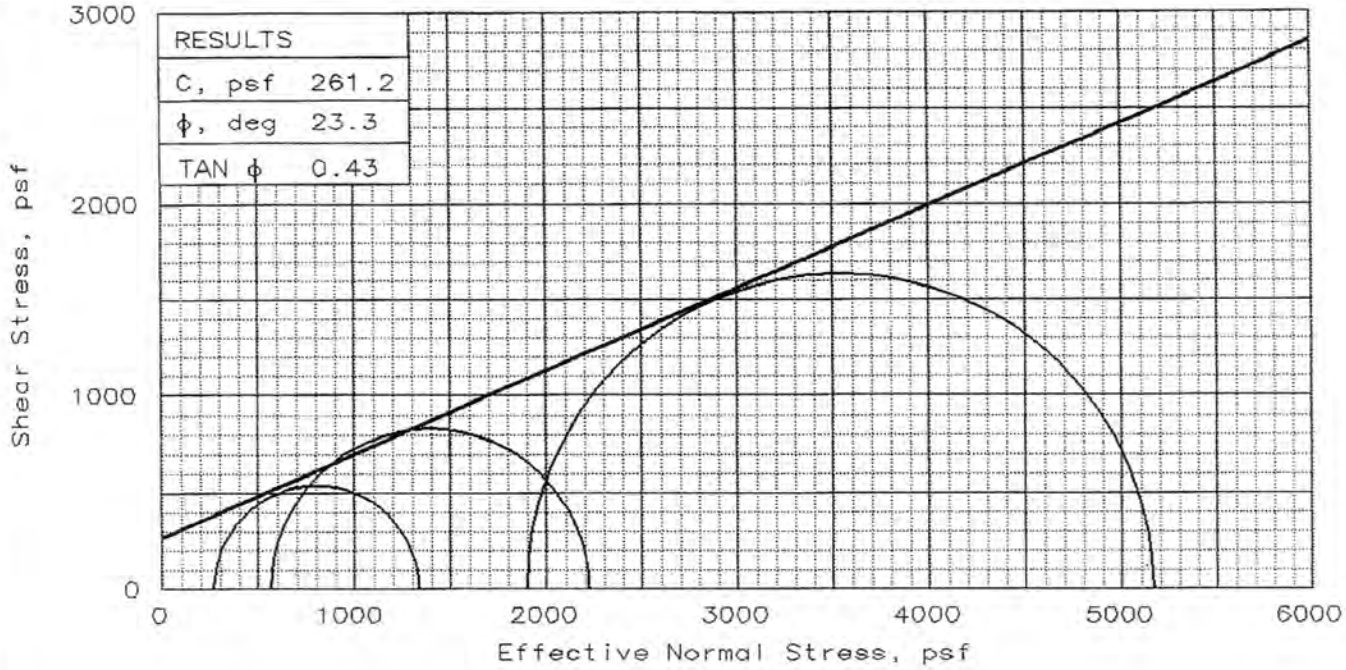


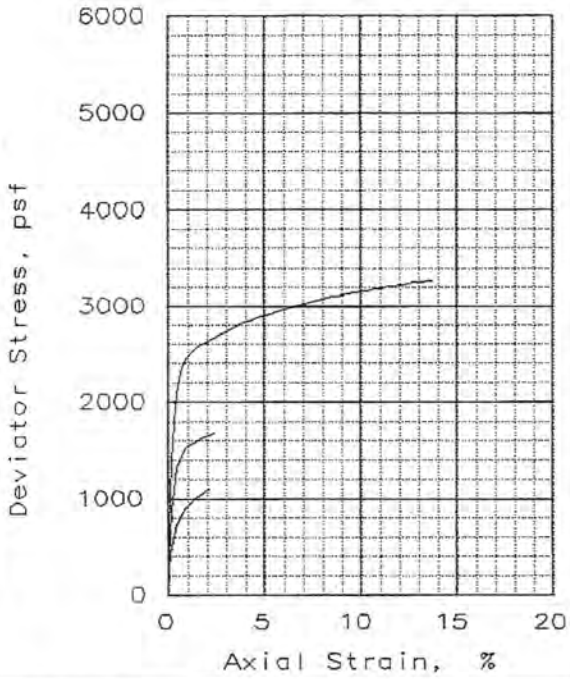

Client: M2 Consultants,Inc.

Project: OCSD

Location: B-2 3A 5.5-8'

File: Project No.: 1202 Plate No: B-10A




| Specimen No.: | | 1 | 2 | 3 |
|---|---|---|---|---|
| INITIAL | WATER CONTENT, % | 29.5 | 29.5 | 29.5 |
| | DRY DENSITY, pcf | 91.7 | 91.7 | 91.7 |
| | SATURATION, % | 95.1 | 95.1 | 95.1 |
| | VOID RATIO | 0.839 | 0.839 | 0.839 |
| | DIAMETER, in | 2.42 | 2.42 | 2.42 |
| | HEIGHT, in | 4.27 | 4.27 | 4.27 |
| AT TEST | WATER CONTENT, % | 29.9 | 29.2 | 28.3 |
| | DRY DENSITY, pcf | 93.4 | 94.5 | 95.8 |
| | SATURATION, % | 100.5 | 100.5 | 100.5 |
| | VOID RATIO | 0.805 | 0.784 | 0.760 |
| | DIAMETER, in | 2.39 | 2.41 | 2.42 |
| | HEIGHT, in | 4.27 | 4.18 | 4.08 |
| Strain rate, in/min | | 0.0030 | 0.0030 | 0.0030 |
| EFF CELL PRESSURE, psf | | 500 | 1000 | 2000 |
| FAIL. STRESS, psf | | 1066 | 1655 | 3266 |
| EXCESS PORE PR., psf | | 226 | 428 | 97 |
| STRAIN, % | | 1.9 | 2.2 | 13.5 |
| ULT. STRESS, psf | | | | |
| EXCESS PORE PR., psf | | | | |
| STRAIN, % | | | | |
| $\bar{\sigma}_1$ FAILURE, psf | | 1340 | 2227 | 5170 |
| $\bar{\sigma}_3$ FAILURE, psf | | 274 | 572 | 1904 |

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: Mod.Cal.
DESCRIPTION: Stiff,black FAT CLAY(CH)w/c-sand inclusions.

SPECIFIC GRAVITY= 2.7
REMARKS:

CLIENT: M2 Consultants,Inc.

PROJECT: OCSD

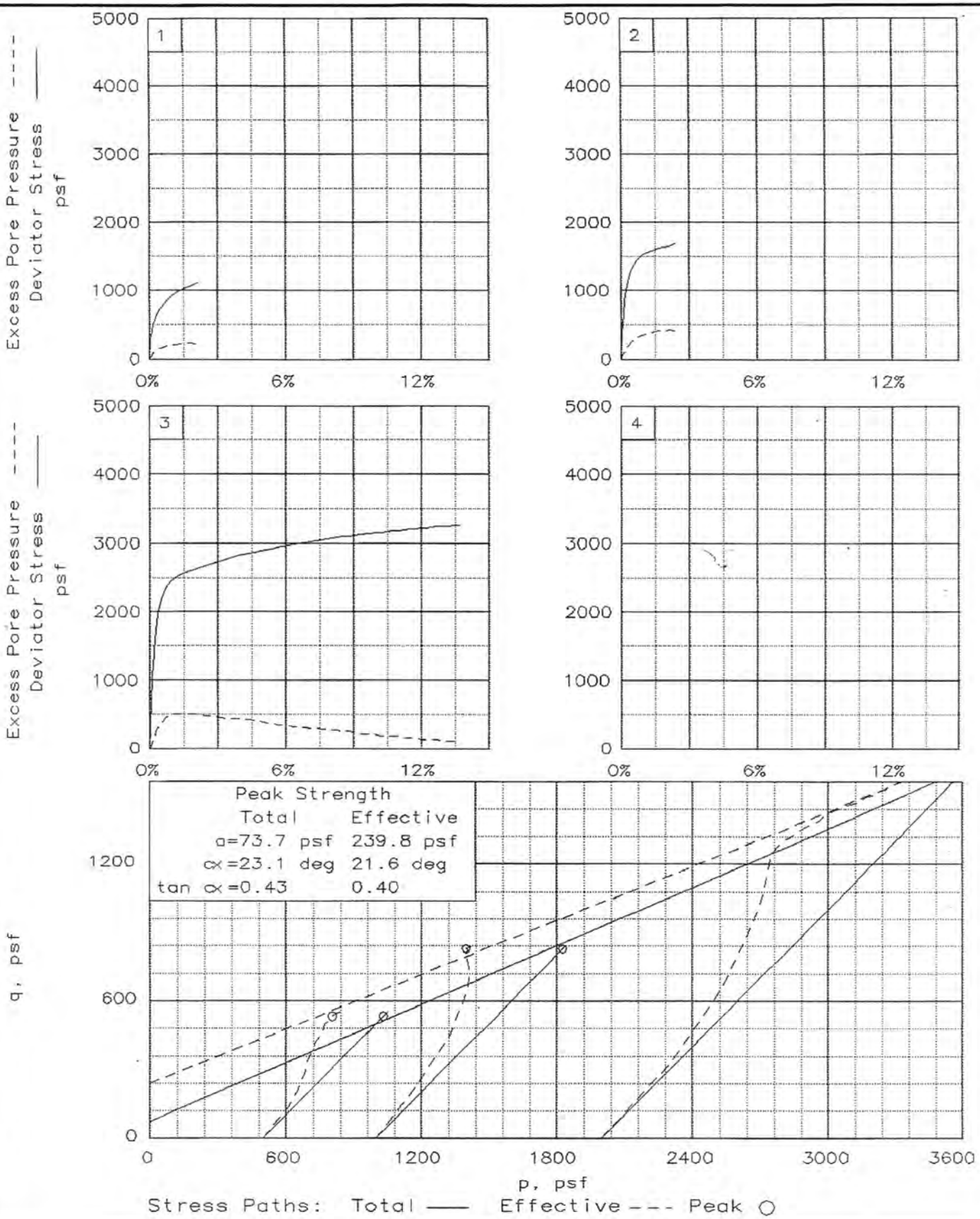
SAMPLE LOCATION: B-3 1C 4'

PROJ. NO.: 1202 DATE: 3-8-12

TRIAXIAL SHEAR TEST REPORT

**Soil Mechanics Lab**

Plate No: B-11

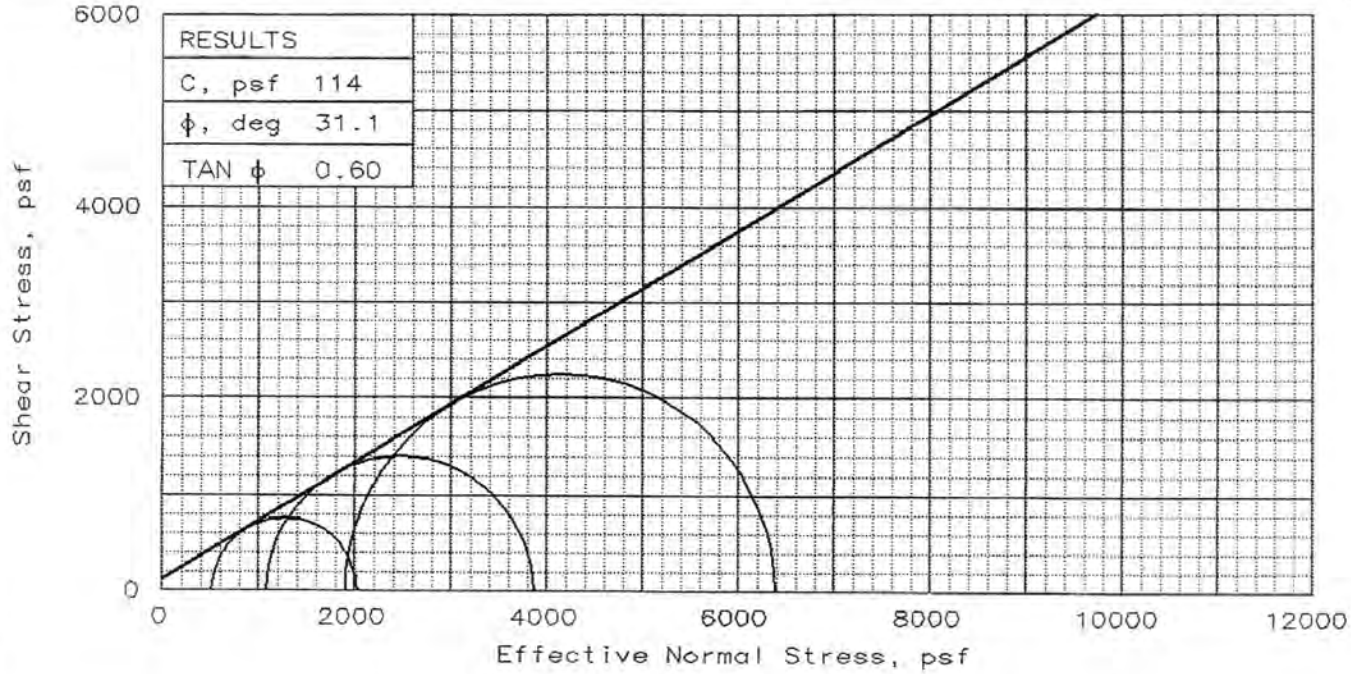


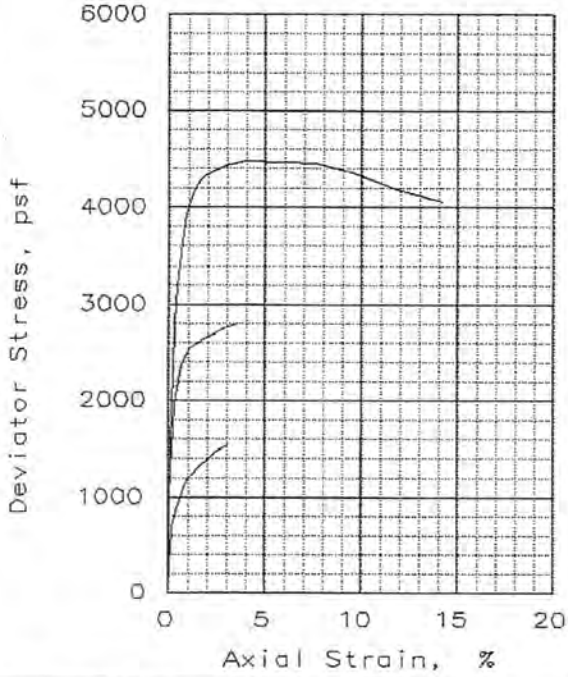

Client: M2 Consultants,Inc.

Project: OCSD

Location: B-3 1C 4'

File: Project No.: 1202 Plate No: B-11A




| | Specimen No.: | 1 | 2 | 3 |
|---|---|---|---|---|
| INITIAL | WATER CONTENT, % | 39.1 | 39.1 | 39.1 |
| | DRY DENSITY, pcf | 81.0 | 81.0 | 81.0 |
| | SATURATION, % | 97.7 | 97.7 | 97.7 |
| | VOID RATIO | 1.082 | 1.082 | 1.082 |
| | DIAMETER, in | 2.42 | 2.42 | 2.42 |
| | HEIGHT, in | 4.60 | 4.60 | 4.60 |
| AT TEST | WATER CONTENT, % | 36.4 | 34.5 | 32.1 |
| | DRY DENSITY, pcf | 85.0 | 87.2 | 90.3 |
| | SATURATION, % | 100.1 | 100.1 | 100.1 |
| | VOID RATIO | 0.983 | 0.932 | 0.866 |
| | DIAMETER, in | 2.36 | 2.36 | 2.37 |
| | HEIGHT, in | 4.60 | 4.46 | 4.30 |
| Strain rate, in/min | | 0.0030 | 0.0030 | 0.0030 |
| EFF CELL PRESSURE, psf | | 1000 | 2000 | 4000 |
| FAIL. STRESS, psf | | 1510 | 2780 | 4487 |
| EXCESS PORE PR., psf | | 475 | 907 | 2088 |
| STRAIN, % | | 2.8 | 3.4 | 4.7 |
| ULT. STRESS, psf | | | | |
| EXCESS PORE PR., psf | | | | |
| STRAIN, % | | | | |
| $\bar{\sigma}_1$ FAILURE, psf | | 2034 | 3873 | 6399 |
| $\bar{\sigma}_3$ FAILURE, psf | | 525 | 1093 | 1912 |

TYPE OF TEST:
CU with Pore Pressures
SAMPLE TYPE: Mod.Cal.
DESCRIPTION: Dense,mottled
very dark grey clayey SAND(SC

SPECIFIC GRAVITY= 2.7
REMARKS:

CLIENT: M2 Consultants,Inc.

PROJECT: OCSD

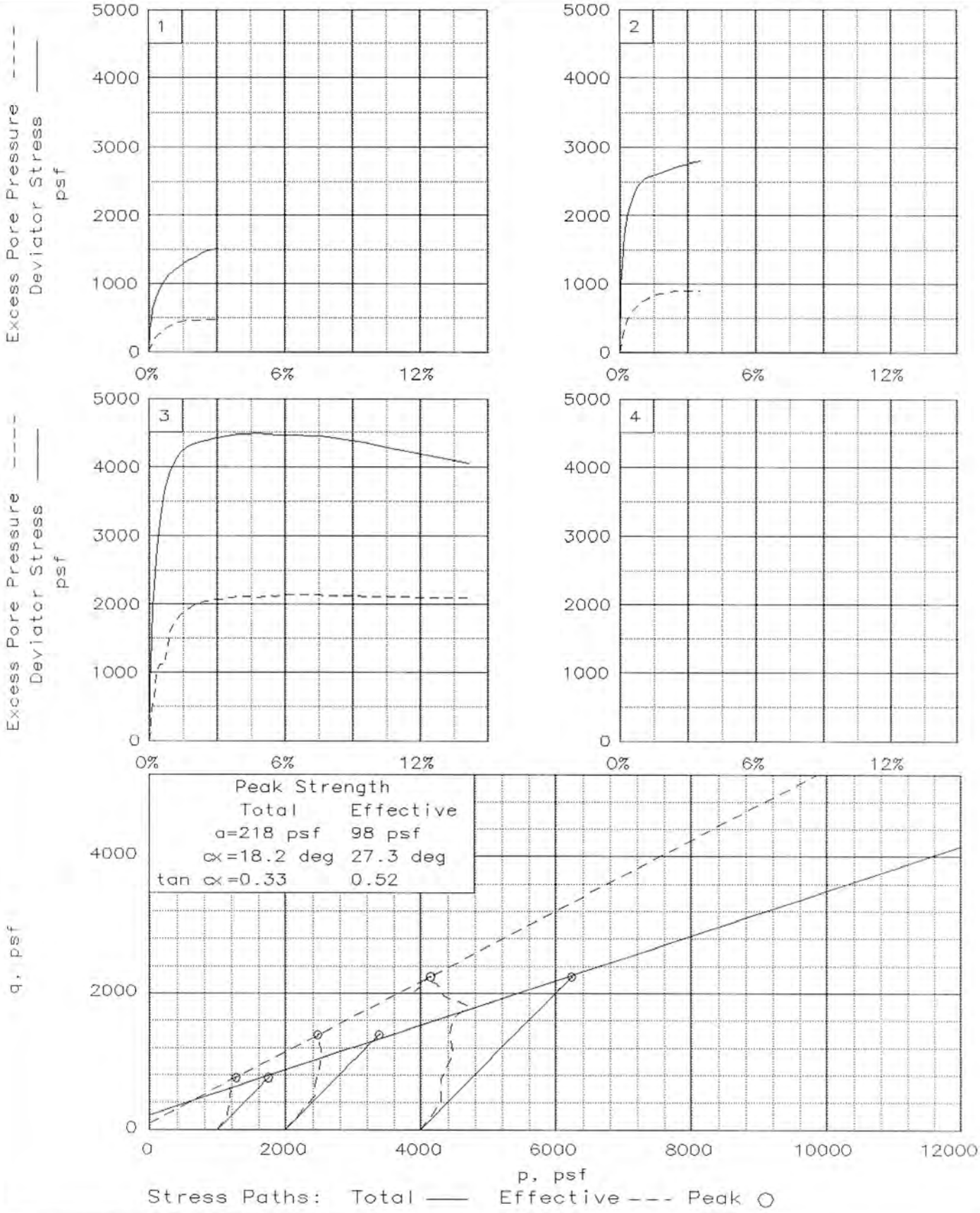
SAMPLE LOCATION: B-4 5C 11'

PROJ. NO.: 1202 DATE: 3-8-12

TRIAXIAL SHEAR TEST REPORT

**Soil Mechanics Lab**

Plate No: B-12




Client: M2 Consultants,Inc.

Project: OCSD

Location: B-4 5C 11'

File: 120244C Project No.: 1202 Plate No: B-12A

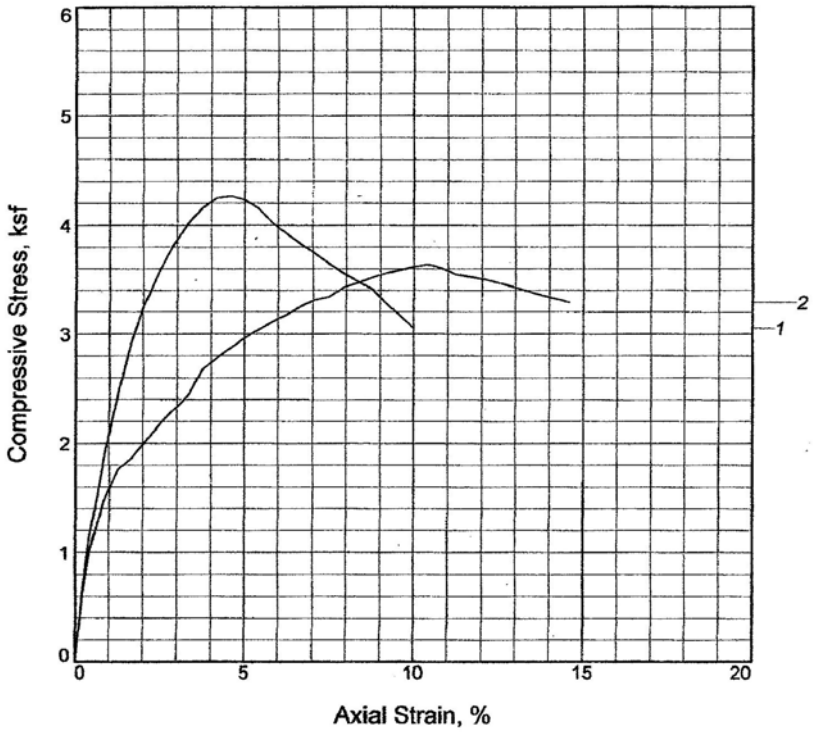
# APPENDIX B

# LABORATORY TESTS

# SITE 3C-2

# PLATES B-13 THROUGH B-23

# UNCONSOLIDATED UNDRAINED TEST




| Sample No. | 1 | 2 | | |
|---|---|---|---|---|
| Fail. Stress, ksf | 4.268 | 3.641 | | |
| Ult. Stress, ksf | | | | |
| Cell pressure, psi | 0.600 | 2.000 | | |
| Strain rate, in./min. | 0.08 | 0.08 | | |
| Water content, % | 34.8 | 18.1 | | |
| Wet density, pcf | 113.9 | 133.6 | | |
| Dry density, pcf | 84.5 | 113.1 | | |
| Saturation, % | 94.5 | 99.9 | | |
| Void ratio | 0.9948 | 0.4898 | | |
| Specimen diameter, in. | 2.42 | 2.42 | | |
| Specimen height, in. | 4.80 | 4.80 | | |
| Height/diameter ratio | 1.99 | 1.99 | | |

**Description:** See remarks

| LL = | PL = | PI = | **Assumed GS=** 2.70 | **Type:** Mod.Cal. |
|---|---|---|---|---|

**Project No.:**

**Date Sampled:**

**Remarks:**
#1/ @ 5.5':Very stiff,dark orange brown clayey SILT(ML/SC)with fine gravel.
#2/ @ 15.5':----------same as 5.5'----------(ML/SC)

**Plate** B-13

**Client:** M2 Consultants,Inc.

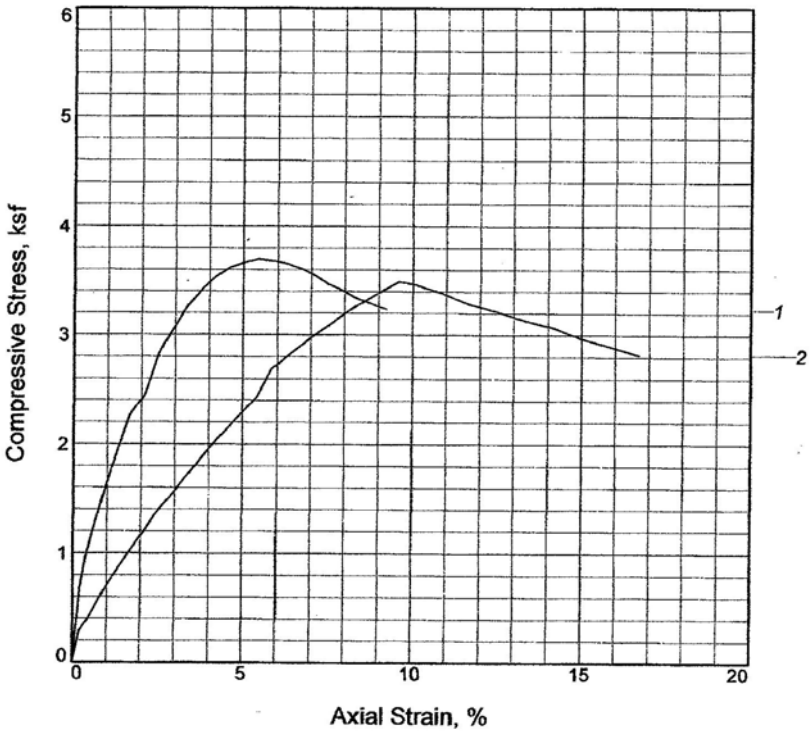
**Project:** OCSD Site 3C-2

**Location:** B-5

**Sample Number:** 1B

UNCONSOLIDATED UNDRAINED TEST
Soil Mechanics Lab
Oakland, California

# UNCONSOLIDATED UNDRAINED TEST




| Sample No. | 1 | 2 | | |
|---|---|---|---|---|
| Fail. Stress, ksf | 3.695 | 3.488 | | |
| Ult. Stress, ksf | | | | |
| Cell pressure, psi | 2.500 | 4.000 | | |
| Strain rate, in./min. | 0.08 | 0.08 | | |
| Water content, % | 21.3 | 15.8 | | |
| Wet density, pcf | 126.5 | 128.1 | | |
| Dry density, pcf | 104.3 | 110.6 | | |
| Saturation, % | 93.2 | 81.5 | | |
| Void ratio | 0.6161 | 0.5234 | | |
| Specimen diameter, in. | 2.42 | 2.42 | | |
| Specimen height, in. | 4.78 | 4.80 | | |
| Height/diameter ratio | 1.98 | 1.99 | | |

**Description:** See remarks

| **LL =** | **PL =** | **PI =** | **Assumed GS=** 2.70 | **Type:** Mod.Cal. |
|---|---|---|---|---|

**Project No.:**

**Date Sampled:**

**Remarks:**
#1/ @ 20.5':Very stiff,dark orange brown clayey SAND(SC)with fine gravel.
#2/ @ 31':----------Same as 20.5'----------(SC).

**Plate** B-14

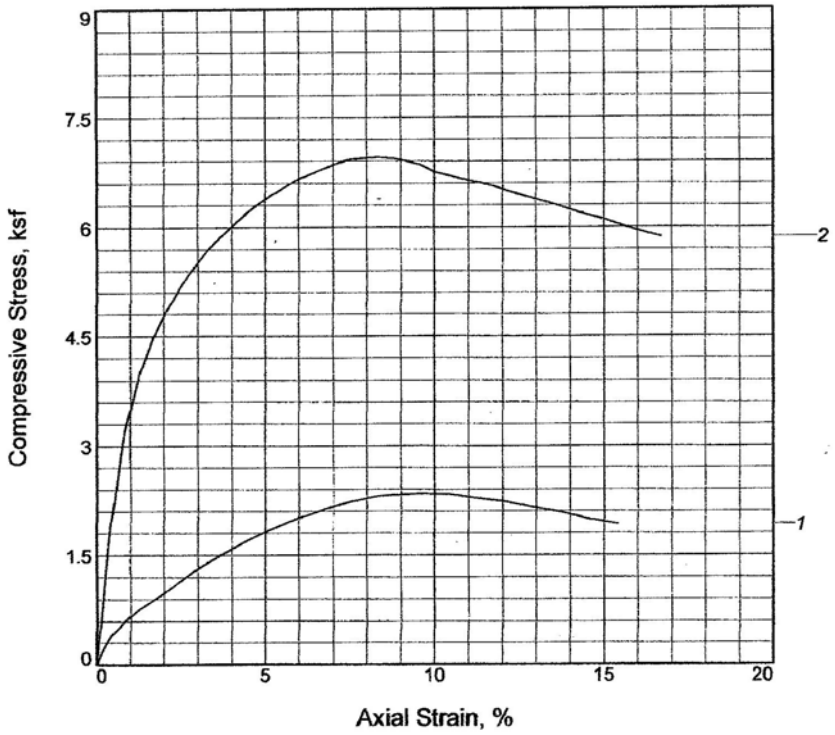
**Client:** M2 Consultants,Inc.

**Project:** OCSD Site 3C-2

**Location:** B-6

UNCONSOLIDATED UNDRAINED TEST
Soil Mechanics Lab
Oakland, California

# UNCONSOLIDATED UNDRAINED TEST




| Sample No. | 1 | 2 | | |
|---|---|---|---|---|
| Fail. Stress, ksf | 2.334 | 6.954 | | |
| Ult. Stress, ksf | | | | |
| Cell pressure, psi | 1.300 | 3.000 | | |
| Strain rate, in./min. | 0.08 | 0.08 | | |
| Water content, % | 22.8 | 15.0 | | |
| Wet density, pcf | 120.9 | 135.6 | | |
| Dry density, pcf | 98.4 | 117.9 | | |
| Saturation, % | 86.4 | 94.2 | | |
| Void ratio | 0.7122 | 0.4298 | | |
| Specimen diameter, in. | 2.42 | 2.42 | | |
| Specimen height, in. | 4.80 | 4.80 | | |
| Height/diameter ratio | 1.99 | 1.99 | | |

**Description:** See remarks.

| **LL =** | **PL =** | **PI =** | **Assumed GS=** 2.70 | **Type:** Mod.Cal. |
|---|---|---|---|---|

**Project No.:**

**Date Sampled:**

**Remarks:**
#1/ @ 11':Dense,dark brown silty fine SAND(SM).
#2/ @ 21':Dense,dk. orange brn. silty SAND{SM)with gravel.

**Plate** B-15

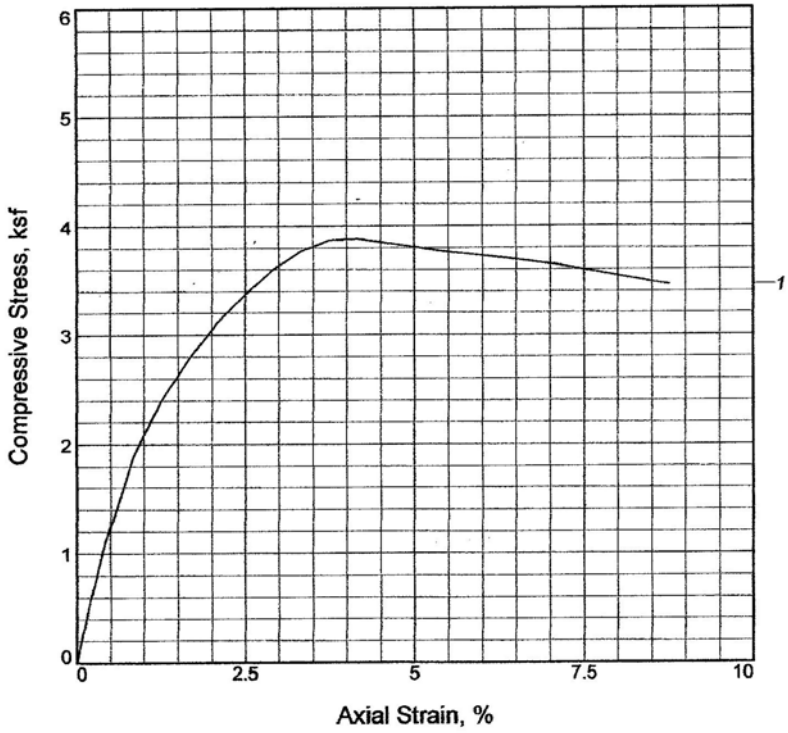
**Client:** M2 Consultants,Inc.

**Project:** OCSD Site 3C-2

**Location:** B-7

UNCONSOLIDATED UNDRAINED TEST
Soil Mechanics Lab
Oakland, California

# UNCONSOLIDATED UNDRAINED TEST




| | | | | |
|---|---|---|---|---|
| Sample No. | 1 | | | |
| Fail. Stress, ksf | 3.883 | | | |
| Ult. Stress, ksf | | | | |
| Cell pressure, psi | 3.500 | | | |
| Strain rate, in./min. | 0.08 | | | |
| Water content, % | 13.1 | | | |
| Dry density, pcf | 110.4 | | | |
| Saturation, % | 83.5 | | | |
| Void ratio | 0.4240 | | | |
| Specimen diameter, in. | 2.42 | | | |
| Specimen height, in. | 4.80 | | | |
| Height/diameter ratio | 1.99 | | | |

**Description:** Very dense,orange brown silty SAND(SM-SC)with gravel.

| | | | | |
|---|---|---|---|---|
| **LL =** | **PL =** | **PI =** | **Assumed GS=** 2.70 | **Type:** Mod.Cal. |

**Project No.:**

**Date Sampled:**

**Remarks:**

**Client:** M2 Consultants,Inc.

**Project:** OCSD Site 3C-2

**Location:** B-10

**Sample Number:** 6C **Depth:** 26'

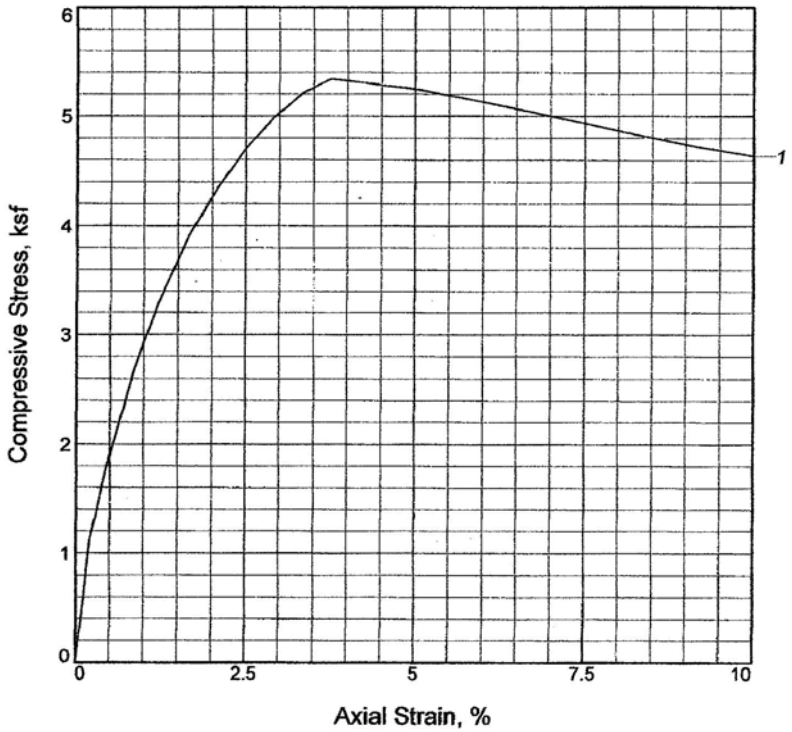
UNCONSOLIDATED UNDRAINED TEST

Soil Mechanics Lab

Oakland, California

**Plate** B-16

# UNCONSOLIDATED UNDRAINED TEST




| | | | | |
|---|---|---|---|---|
| Sample No. | 1 | | | |
| Fail. Stress, ksf | 5.342 | | | |
| Ult. Stress, ksf | | | | |
| Cell pressure, psi | 1.500 | | | |
| Strain rate, in./min. | 0.08 | | | |
| Water content, % | 17.0 | | | |
| Wet density, pcf | 129.0 | | | |
| Dry density, pcf | 110.3 | | | |
| Saturation, % | 86.8 | | | |
| Void ratio | 0.5287 | | | |
| Specimen diameter, in. | 2.42 | | | |
| Specimen height, in. | 4.80 | | | |
| Height/diameter ratio | 1.99 | | | |

**Description:** Very stiff,very dark purplish grey LEAN CLAY(CL)

| | | | | |
|---|---|---|---|---|
| **LL =** | **PL =** | **PI =** | **Assumed GS=** 2.70 | **Type:** Mod.Cal. |

**Project No.:**

**Date Sampled:**

**Remarks:**

**Client:** M2 Consultants,Inc.

**Project:** OCSD Site 3C-2

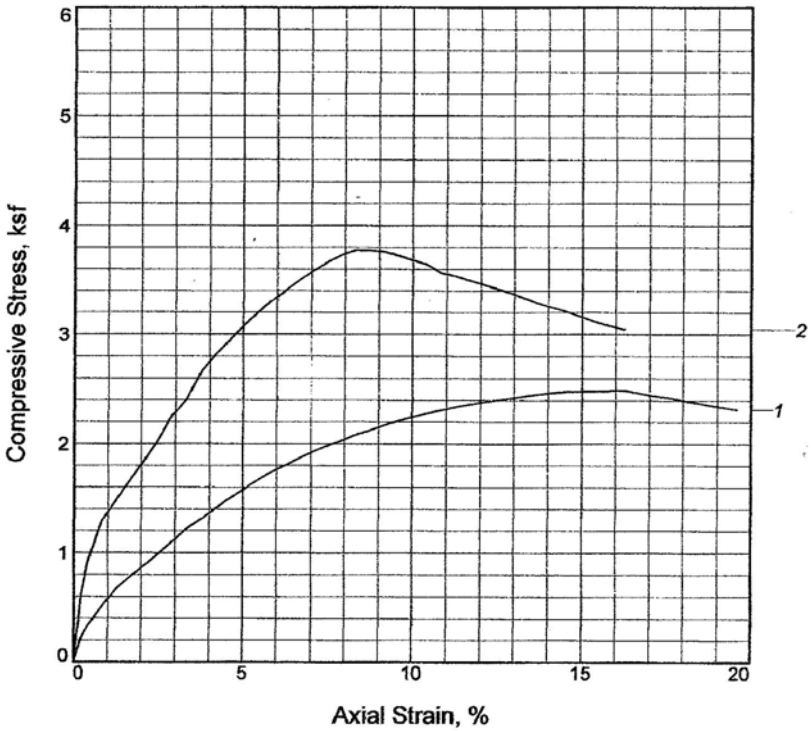
**Location:** B-11

**Sample Number:** 2C **Depth:** 11'

UNCONSOLIDATED UNDRAINED TEST
Soil Mechanics Lab
Oakland, California

**Plate** B-17

# UNCONSOLIDATED UNDRAINED TEST




| Sample No. | 1 | 2 | | |
|---|---|---|---|---|
| Fail. Stress, ksf | 2.483 | 3.769 | | |
| Ult. Stress, ksf | | | | |
| Cell pressure, psi | 1.300 | 2.000 | | |
| Strain rate, in./min. | 0.08 | 0.08 | | |
| Water content, % | 23.6 | 26.6 | | |
| Wet density, pcf | 104.2 | 115.6 | | |
| Dry density, pcf | 84.3 | 91.3 | | |
| Saturation, % | 63.8 | 84.9 | | |
| Void ratio | 1.0003 | 0.8452 | | |
| Specimen diameter, in. | 2.42 | 2.42 | | |
| Specimen height, in. | 4.80 | 4.80 | | |
| Height/diameter ratio | 1.99 | 1.99 | | |

**Description:** See remarks

| LL = | PL = | PI = | **Assumed GS=** 2.70 | **Type:** Mod.Cal. |
|---|---|---|---|---|

**Project No.:**

**Date Sampled:**

**Remarks:**
#1/ @ 12.5':Dense,orange brown clayey SAND(SC)
#2/ @ 16.0':----------same as 12.5'-------(SC)

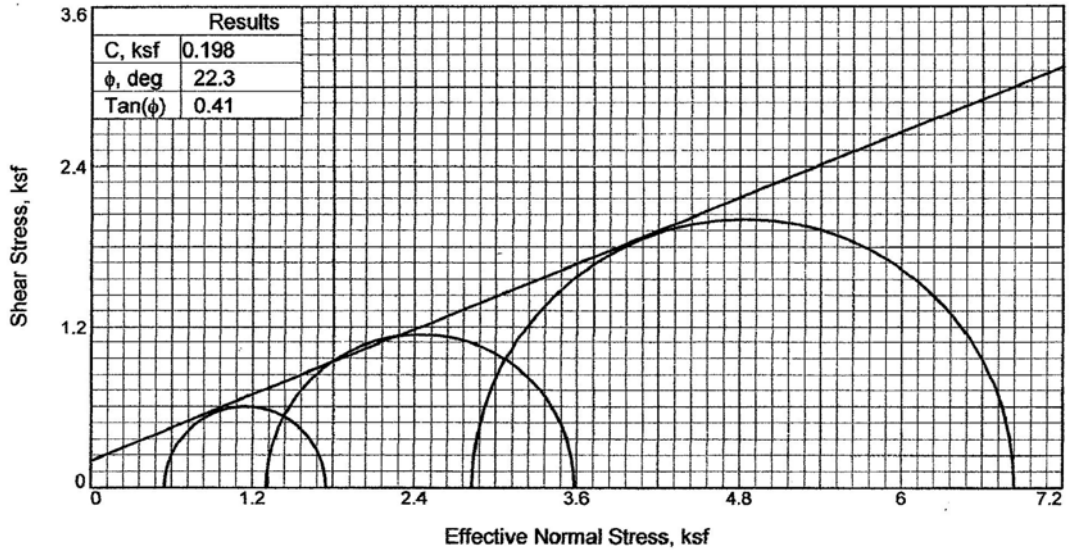
**Plate** B-18

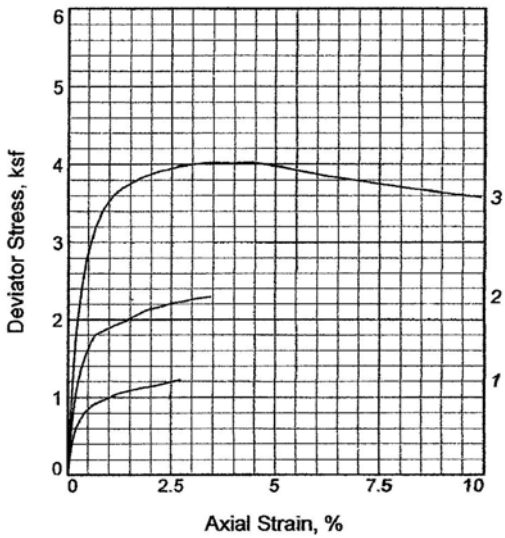
**Client:** M2 Consultants,Inc.

**Project:** OCSD Site 3C-2

**Location:** B-12

UNCONSOLIDATED UNDRAINED TEST
Soil Mechanics Lab
Oakland, California




| Sample No. | | 1 | 2 | 3 |
|---|---|---|---|---|
| Initial | Water Content, % | 23.7 | 23.7 | 23.7 |
| | Dry Density, pcf | 103.3 | 103.3 | 103.3 |
| | Saturation, % | 101.4 | 101.4 | 101.4 |
| | Void Ratio | 0.6316 | 0.6316 | 0.6316 |
| | Diameter, in. | 2.42 | 2.42 | 2.42 |
| | Height, in. | 4.80 | 4.80 | 4.80 |
| At Test | Water Content, % | 23.0 | 22.3 | 21.4 |
| | Dry Density, pcf | 104.3 | 105.6 | 107.2 |
| | Saturation, % | 100.9 | 101.0 | 101.0 |
| | Void Ratio | 0.6153 | 0.5963 | 0.5728 |
| | Diameter, in. | 2.40 | 2.42 | 2.45 |
| | Height, in. | 4.80 | 4.67 | 4.51 |
| Strain rate, in./min. | | 0.00 | 0.00 | 0.00 |
| Eff. Cell Pressure, ksf | | 1.000 | 2.000 | 4.000 |
| Fail. Stress, ksf | | 1.198 | 2.280 | 4.014 |
| Excess Pore Pr., ksf | | 0.453 | 0.698 | 1.173 |
| Strain, % | | 2.5 | 3.2 | 3.3 |
| Ult. Stress, ksf | | | | |
| Excess Pore Pr., ksf | | | | |
| Strain, % | | | | |
| $\bar{\sigma}_1$ Failure, ksf | | 1.745 | 3.583 | 6.841 |
| $\bar{\sigma}_3$ Failure, ksf | | 0.547 | 1.302 | 2.827 |

**Type of Test:**
CU with Pore Pressures

**Sample Type:** Mod.Cal.

**Description:** Dark,purplish grey LEAN CLAY(CL).

**LL=** 42 **PL=** 18 **PI=** 24

**Assumed Specific Gravity=** 2.70

**Remarks:**

**Client:** M2 Consultants,Inc.

**Project:** OCSD Site 3C-2

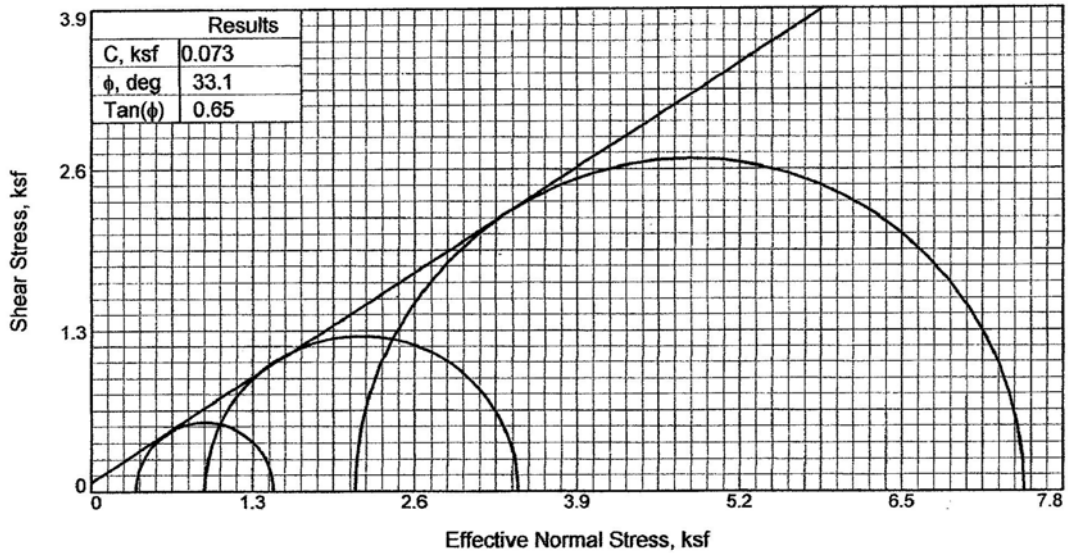
**Location:** B-11

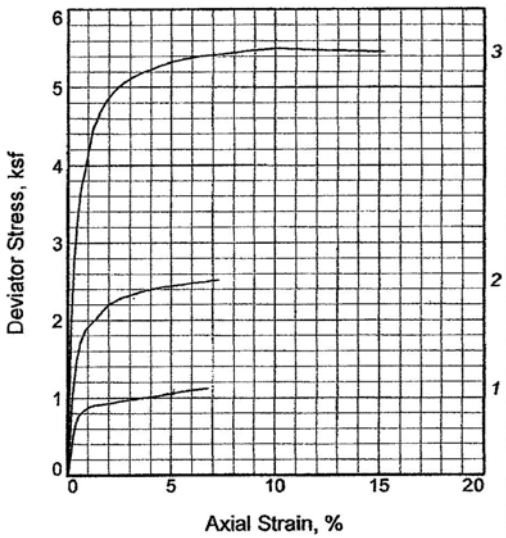
**Sample Number:** 4B **Depth:** 20.5

**Proj. No.:** **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT
Soil Mechanics Lab
Oakland, California

**Plate** B-19




| Sample No. | | 1 | 2 | 3 |
|---|---|---|---|---|
| Initial | Water Content, % | 23.9 | 23.9 | 23.9 |
| | Dry Density, pcf | 94.7 | 94.7 | 94.7 |
| | Saturation, % | 82.8 | 82.8 | 82.8 |
| | Void Ratio | 0.7803 | 0.7803 | 0.7803 |
| | Diameter, in. | 2.42 | 2.42 | 2.42 |
| | Height, in. | 3.72 | 3.72 | 3.72 |
| At Test | Water Content, % | 28.2 | 26.8 | 25.6 |
| | Dry Density, pcf | 95.7 | 97.8 | 99.6 |
| | Saturation, % | 99.9 | 99.9 | 99.9 |
| | Void Ratio | 0.7612 | 0.7242 | 0.6917 |
| | Diameter, in. | 2.40 | 2.46 | 2.53 |
| | Height, in. | 3.72 | 3.47 | 3.22 |
| Strain rate, in./min. | | 0.00 | 0.00 | 0.00 |
| Eff. Cell Pressure, ksf | | 1.000 | 2.000 | 4.000 |
| Fail. Stress, ksf | | 1.106 | 2.511 | 5.363 |
| Excess Pore Pr., ksf | | 0.640 | 1.086 | 1.878 |
| Strain, % | | 6.5 | 6.9 | 5.6 |
| Ult. Stress, ksf | | | | |
| Excess Pore Pr., ksf | | | | |
| Strain, % | | | | |
| $\bar{\sigma}_1$ Failure, ksf | | 1.466 | 3.424 | 7.485 |
| $\bar{\sigma}_3$ Failure, ksf | | 0.360 | 0.914 | 2.122 |

**Type of Test:**
CU with Pore Pressures

**Sample Type:** Mod.Cal.

**Description:** Stiff,very daek brown sandy CLAY(CL)

**Assumed Specific Gravity=** 2.70

**Remarks:**

**Client:** M2 Consultants,Inc.

**Project:** OCSD Site 3C-2

**Location:** B-12

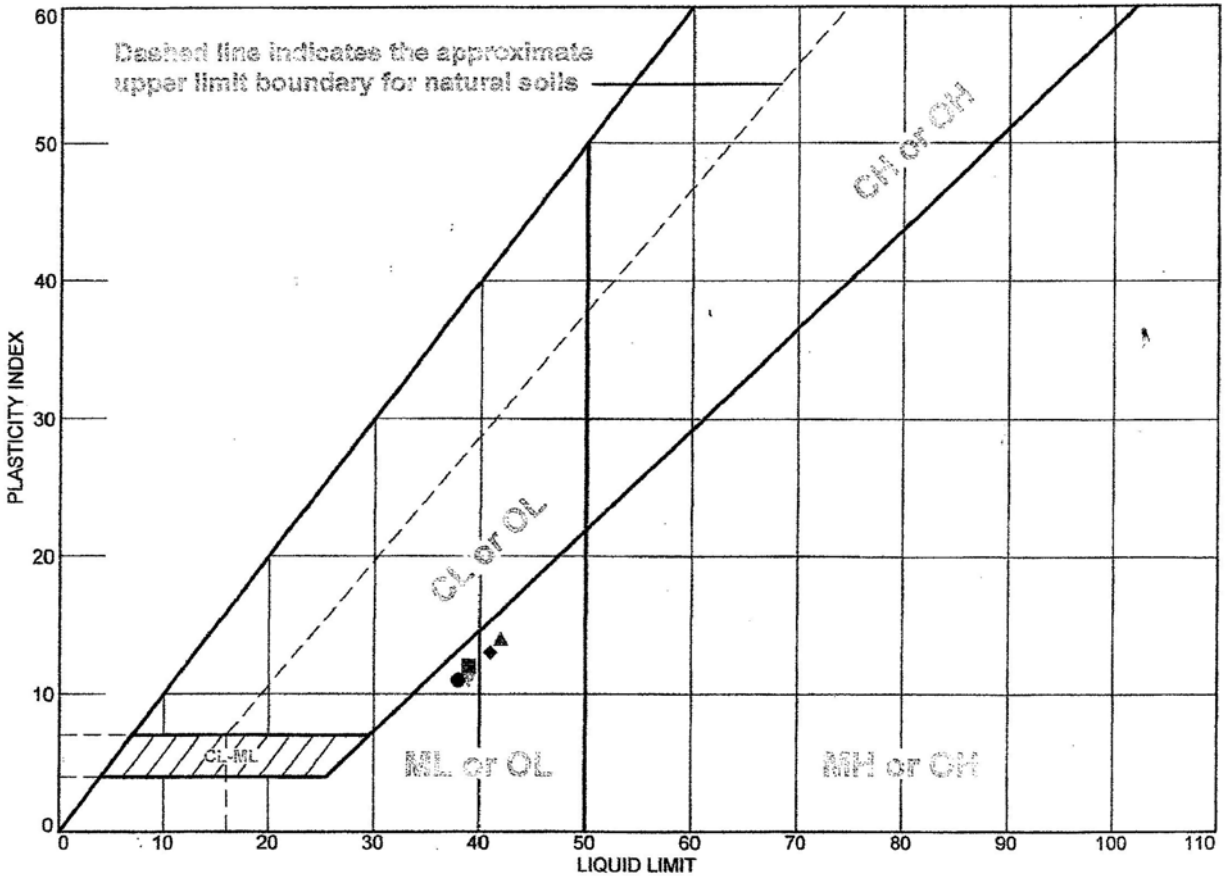
**Sample Number:** 1C **Depth:** 5'

**Proj. No.:** **Date Sampled:**

TRIAXIAL SHEAR TEST REPORT
Soil Mechanics Lab
Oakland, California

**Plate** B-20

# LIQUID AND PLASTIC LIMITS TEST REPORT




| | MATERIAL DESCRIPTION | LL | PL | PI | %<#40 | %<#200 | USCS |
|---|---|---|---|---|---|---|---|
| ● | Dark orange brown clayey SILT. | 38 | 27 | 11 | | | ML |
| ■ | Dark,orange brown clayey SILT with gravel. | 39 | 27 | 12 | | | ML |
| ▲ | Dark,orange brown clayey SILT with gravel. | 42 | 28 | 14 | | | ML |
| ◆ | Dark,orange brown clayey SILT with gravel. | 41 | 28 | 13 | | | ML |
| ▽ | Dark,orange brown clayey SILT with gravel. | 39 | 28 | 11 | | | ML |

**Project No.** **Client:** M2 Consultants,Inc.

**Project:** OCSD Site 3C-2

● **Location:** B-5 **Depth:** 5.5' **Sample Number:** 1B
■ **Location:** B-5 **Depth:** 11' **Sample Number:** 2C
▲ **Location:** B-5 **Depth:** 21' **Sample Number:** 4C
◆ **Location:** B-6 **Depth:** 10.5 **Sample Number:** 2C
▽ **Location:** B-7 **Depth:** 11' **Sample Number:** 2C

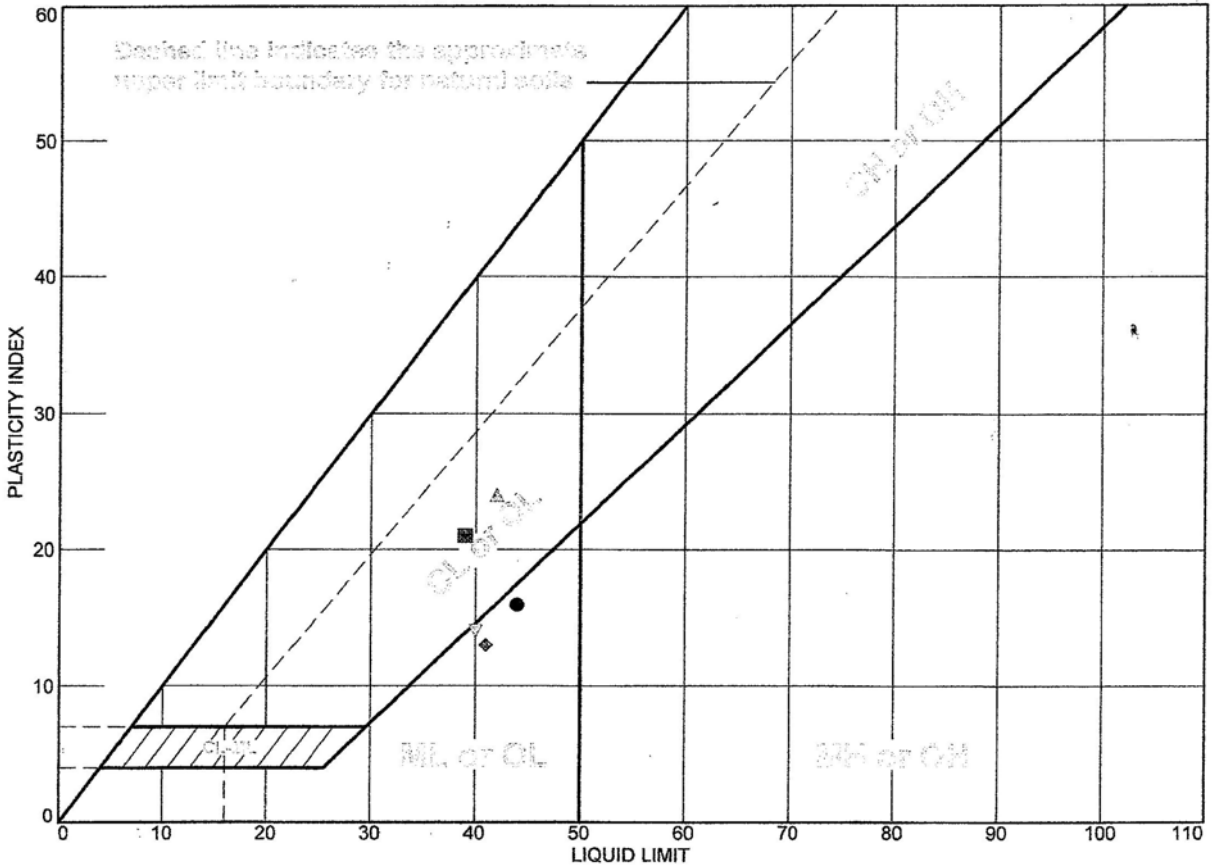
**Remarks:**

**Soil Mechanics Lab**

**Oakland, California**

**Plate** B-21

# LIQUID AND PLASTIC LIMITS TEST REPORT




| | MATERIAL DESCRIPTION | LL | PL | PI | %<#40 | %<#200 | USCS |
|---|---|---|---|---|---|---|---|
| ● | Dark,purplish grey LEAN CLAY.Trace gravel inclusions. | 44 | 28 | 16 | | | ML |
| ■ | Dark,purplish grey LEAN CLAY.Trace gravel incusions. | 39 | 18 | 21 | | | CL |
| ▲ | Dark,purplish grey LEAN CLAY(CL). | 42 | 18 | 24 | | | CL |
| ◆ | Dark,orange brown clayey SILT with gravel.. | 41 | 28 | 13 | | | ML |
| ▽ | Dark,orange brown clayey SILT with gravel. | 40 | 26 | 14 | | | ML |

**Project No.** **Client:** M2 Consultants,Inc.

**Project:** OCSD Site 3C-2

● **Location:** B-8 **Depth:** 5.5' **Sample Number:** 1B
■ **Location:** B-11 **Depth:** 11' **Sample Number:** 2C
▲ **Location:** B-11 **Depth:** 20.5 **Sample Number:** 4B
◆ **Location:** B-12 **Depth:** 12' **Sample Number:** 3A
▽ **Location:** B-12 **Depth:** 16.5 **Sample Number:** 3A

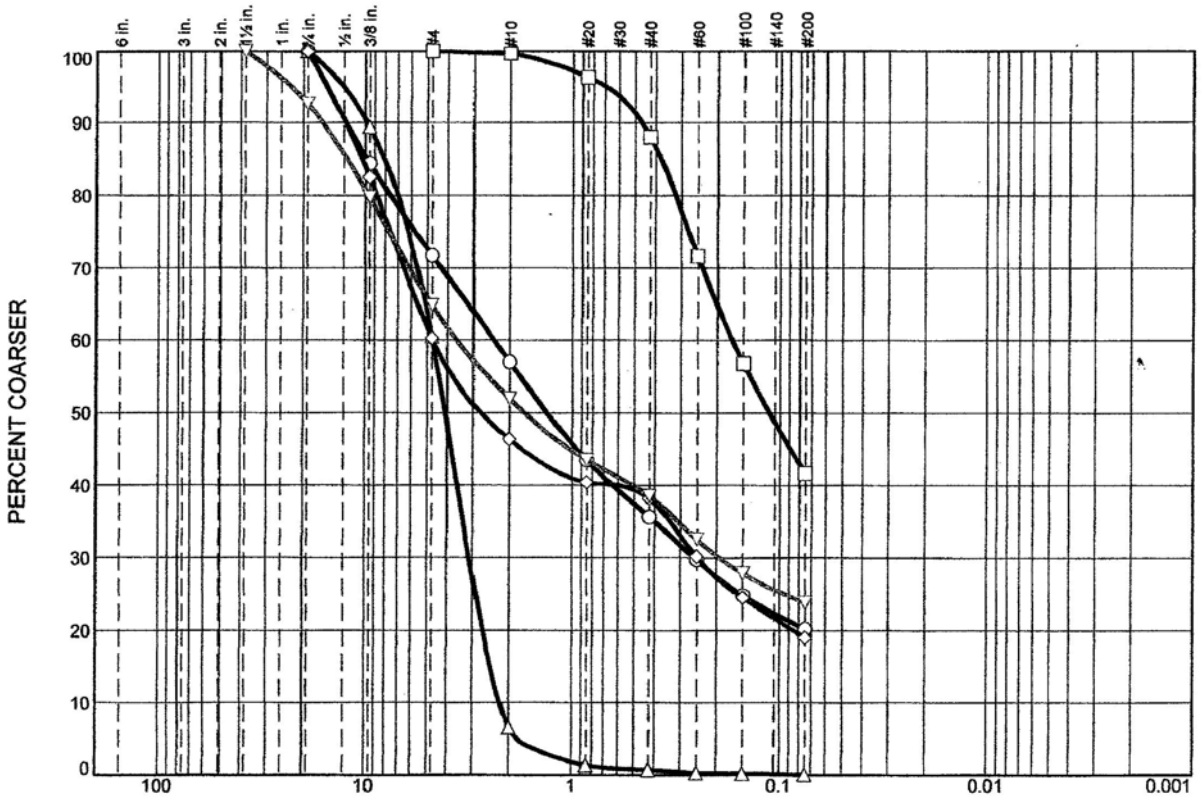
**Remarks:**

**Soil Mechanics Lab**

**Oakland, California**

**Plate** B-22

# Particle Size Distribution Report




| | % +3" | % Gravel | | % Sand | | | % Fines | |
|---|---|---|---|---|---|---|---|---|
| | | Coarse | Fine | Coarse | Medium | Fine | Silt | Clay |
| ○ | 0.0 | 0.0 | 28.2 | 14.8 | 21.3 | 15.5 | 20.2 | |
| □ | 0.0 | 0.0 | 0.0 | 0.3 | 11.7 | 46.3 | 41.7 | |
| △ | 0.0 | 0.0 | 39.5 | 53.9 | 5.9 | 0.7 | 0.0 | |
| ◇ | 0.0 | 0.0 | 39.8 | 13.8 | 8.2 | 19.1 | 19.1 | |
| ▽ | 0.0 | 7.4 | 27.8 | 12.9 | 13.4 | 14.7 | 23.8 | |

## SOIL DATA

| SYMBOL | SOURCE | SAMPLE NO. | DEPTH (ft.) | Material Description | USCS |
|---|---|---|---|---|---|
| ○ | | B-5 3B | 15.5' | Dark,orange brown clayey SAND with gravel. | SM |
| □ | | B-7 2C | 11' | Dark brown silty fine SAND. | SM |
| △ | | B-7 9B | 40.5 | Dark grey fine GRAVEL. | GP |
| ◇ | | B-8 2G | 11' | Dark orange brown silty SAND & GRAVEL. | SM-GM |
| ▽ | | B-8 4C | 26' | Dark,orange brown silty SAND with gravel. | SM |

**Soil Mechanics Lab**

**Oakland, California**

**Client:** M2 Consultants,Inc.
**Project:** OCSD Site 3C-2
**Project No.:**
**Plate** B-23

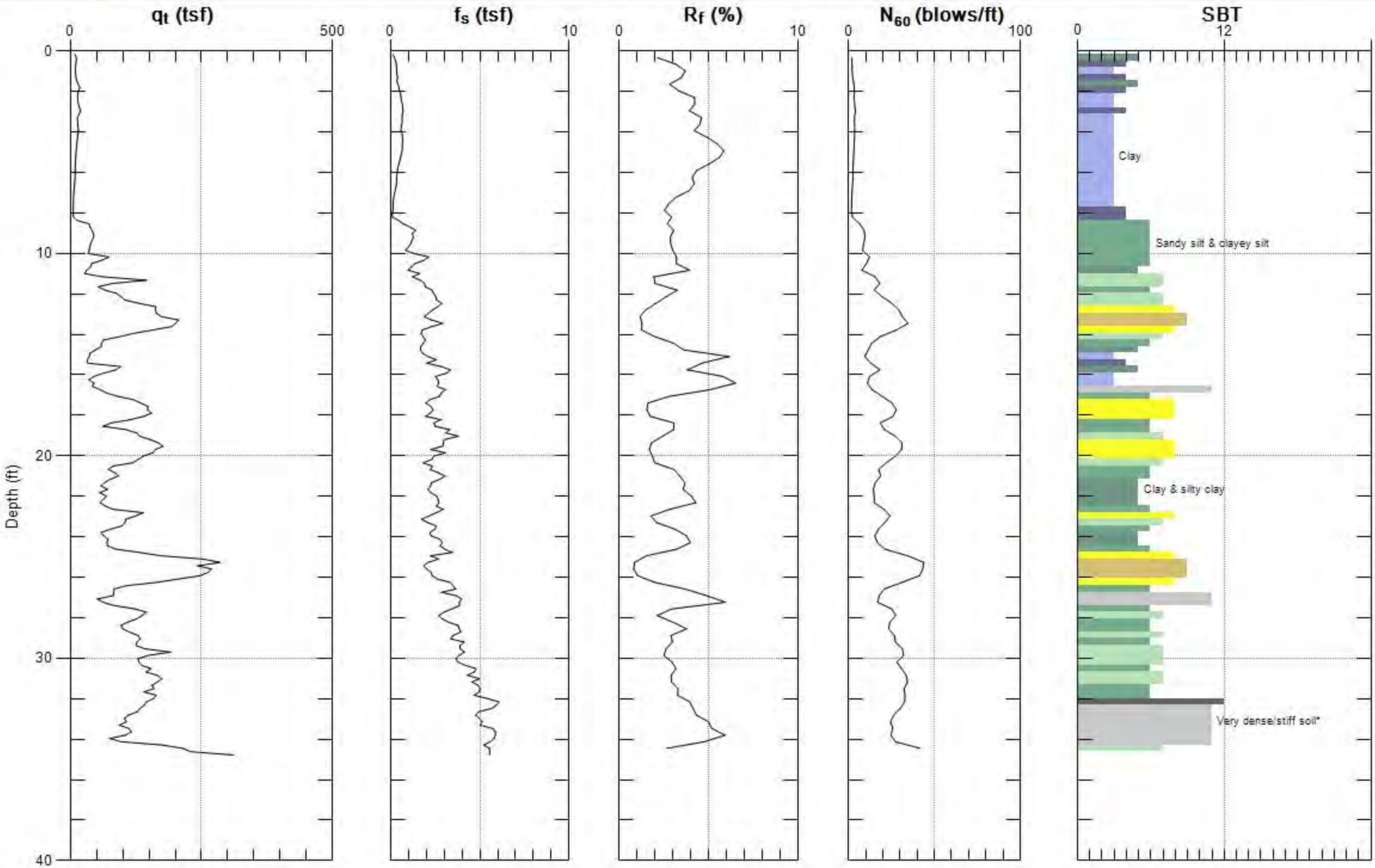
# APPENDIX C

# CONEPENETRATION TESTS

# M2 CONSULTANTS

Site: OCSD
Sounding: CPT-1

Engineer: A.JOHNS
Date: 2/20/2012 02:16



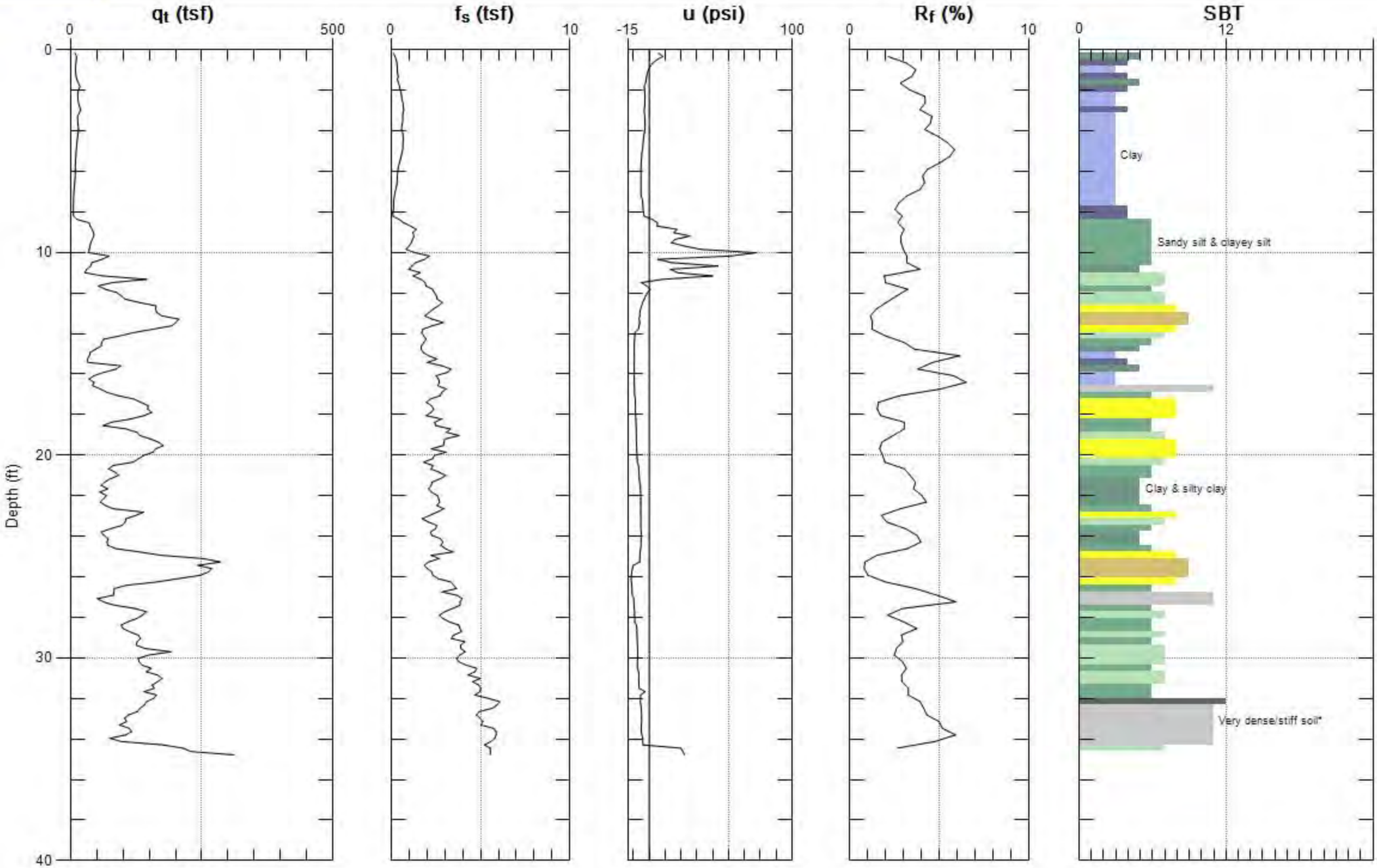

Max. Depth: 34.777 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

# M2 CONSULTANTS

Site: OCSD
Sounding: CPT-1

Engineer: A.JOHNS
Date: 2/20/2012 02:16



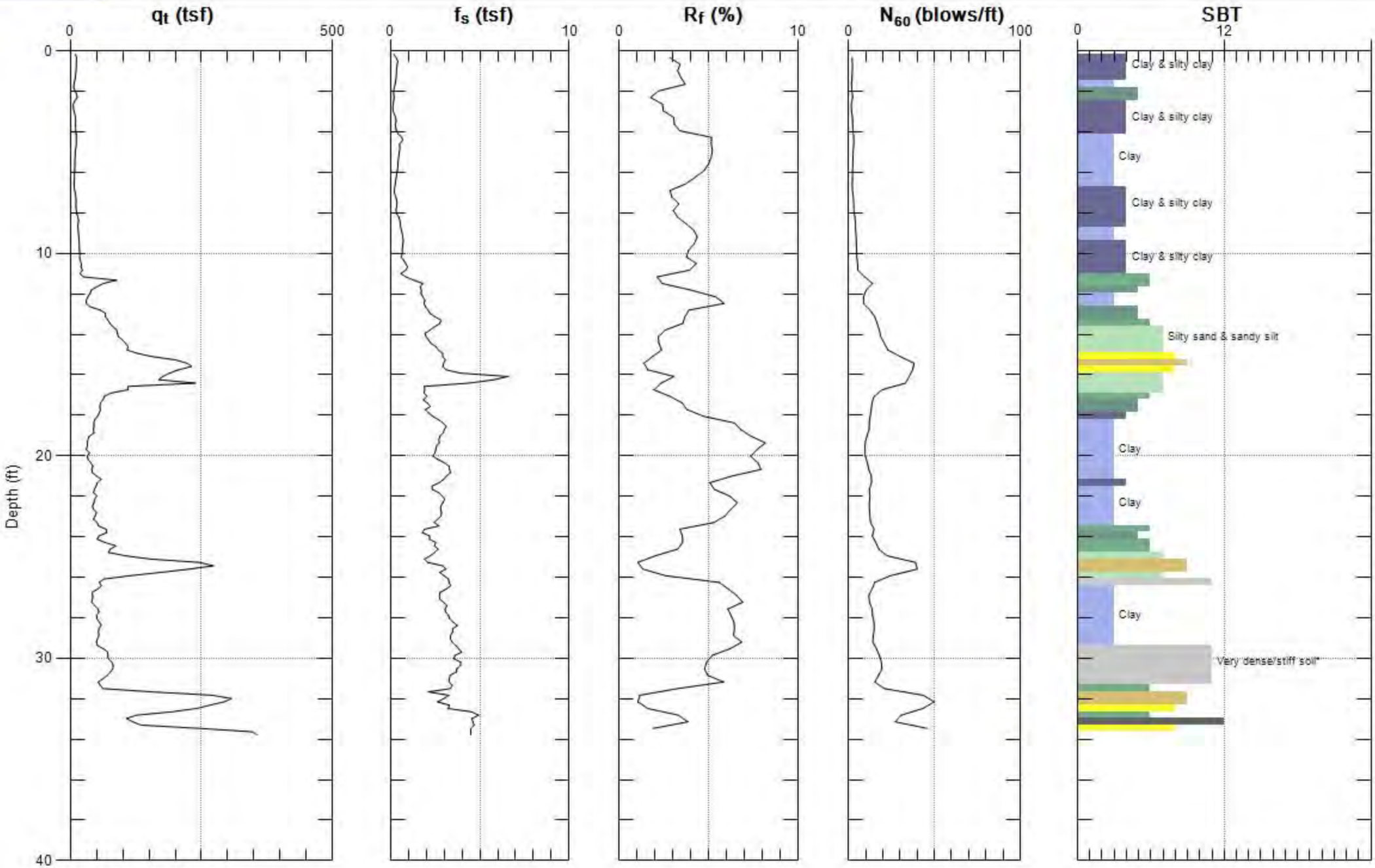

Max. Depth: 34.777 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

# M2 CONSULTANTS

Site: OCSD
Sounding: CPT-2

Engineer: A.JOHNS
Date: 2/20/2012 01:21



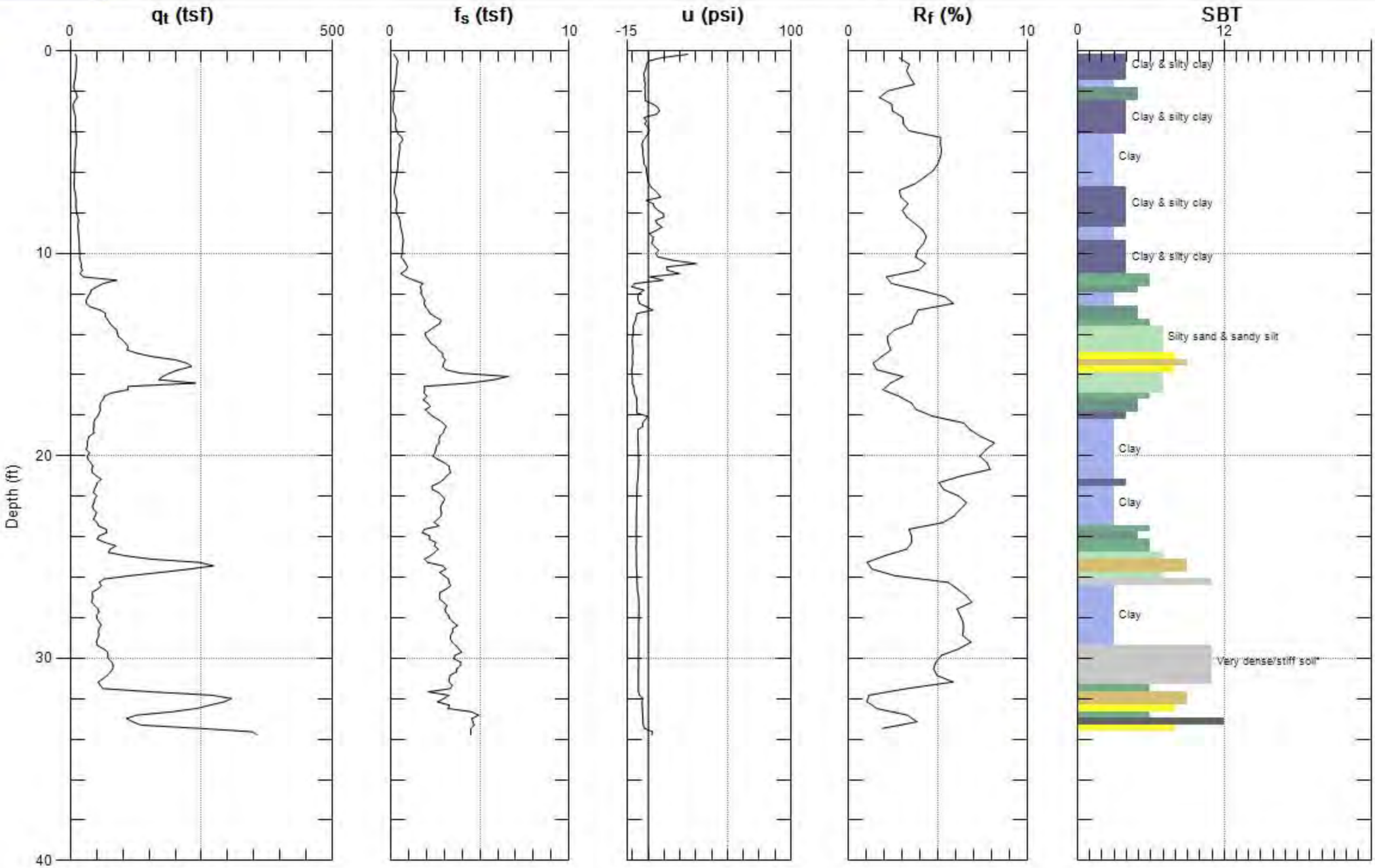

Max. Depth: 33.793 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

# M2 CONSULTANTS

Site: OCSD
Sounding: CPT-2

Engineer: A.JOHNS
Date: 2/20/2012 01:21



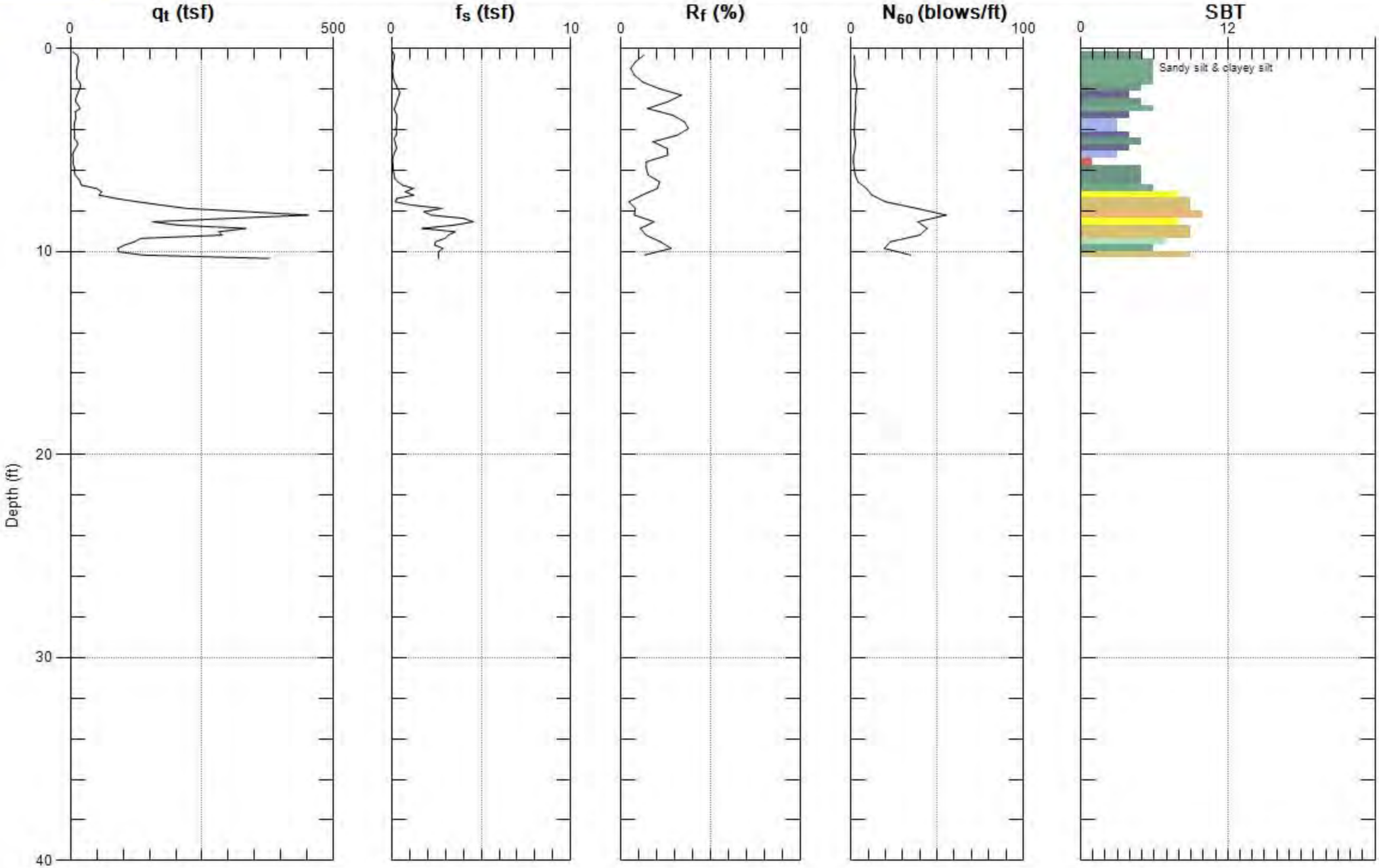

Max. Depth: 33.793 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

# M2 CONSULTANTS

Site: OCSD
Sounding: CPT-3

Engineer: A.JOHNS
Date: 2/20/2012 12:28



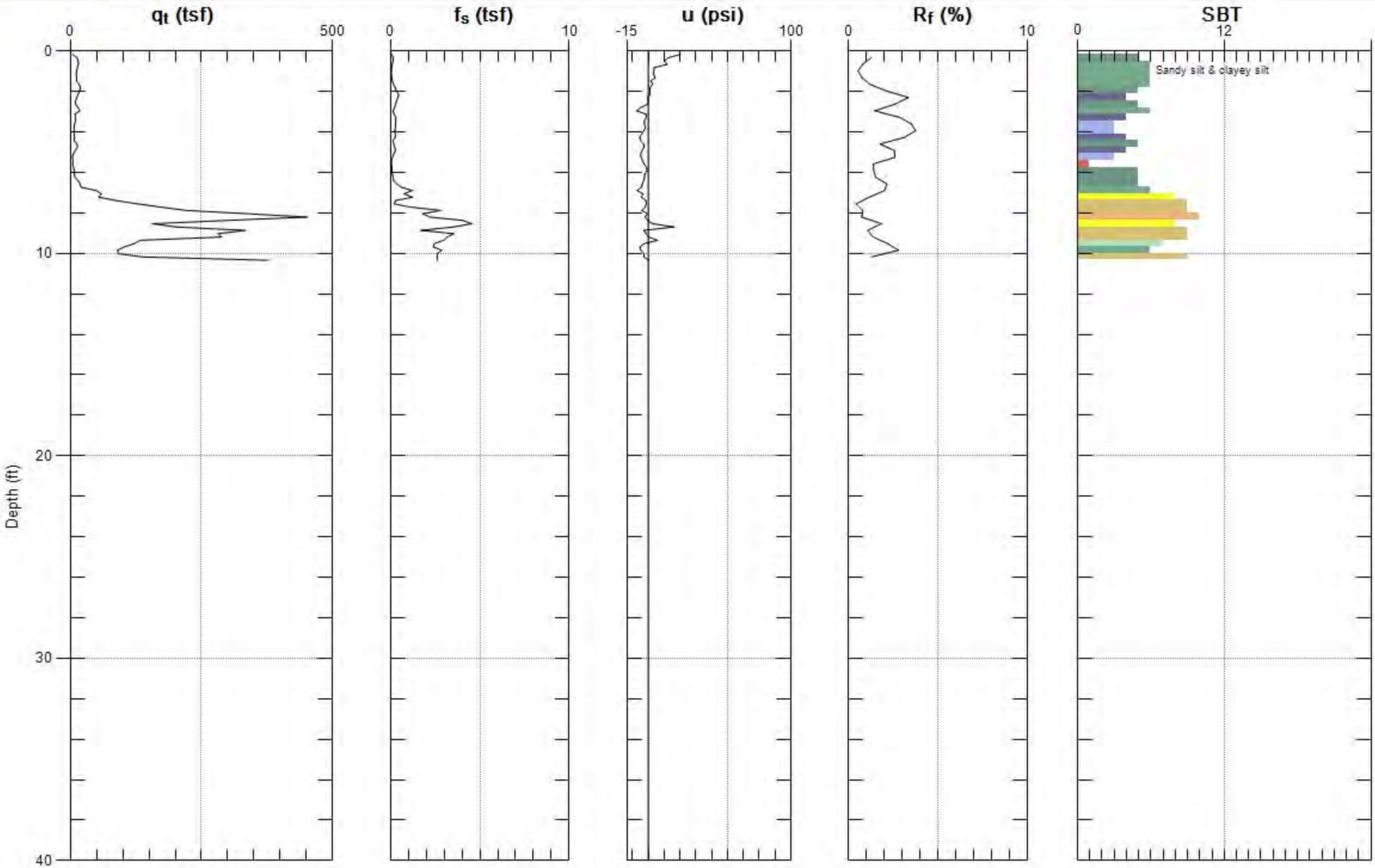

Max. Depth: 10.335 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

# M2 CONSULTANTS

Site: OCSD
Sounding: CPT-3

Engineer: A.JOHNS
Date: 2/20/2012 12:28



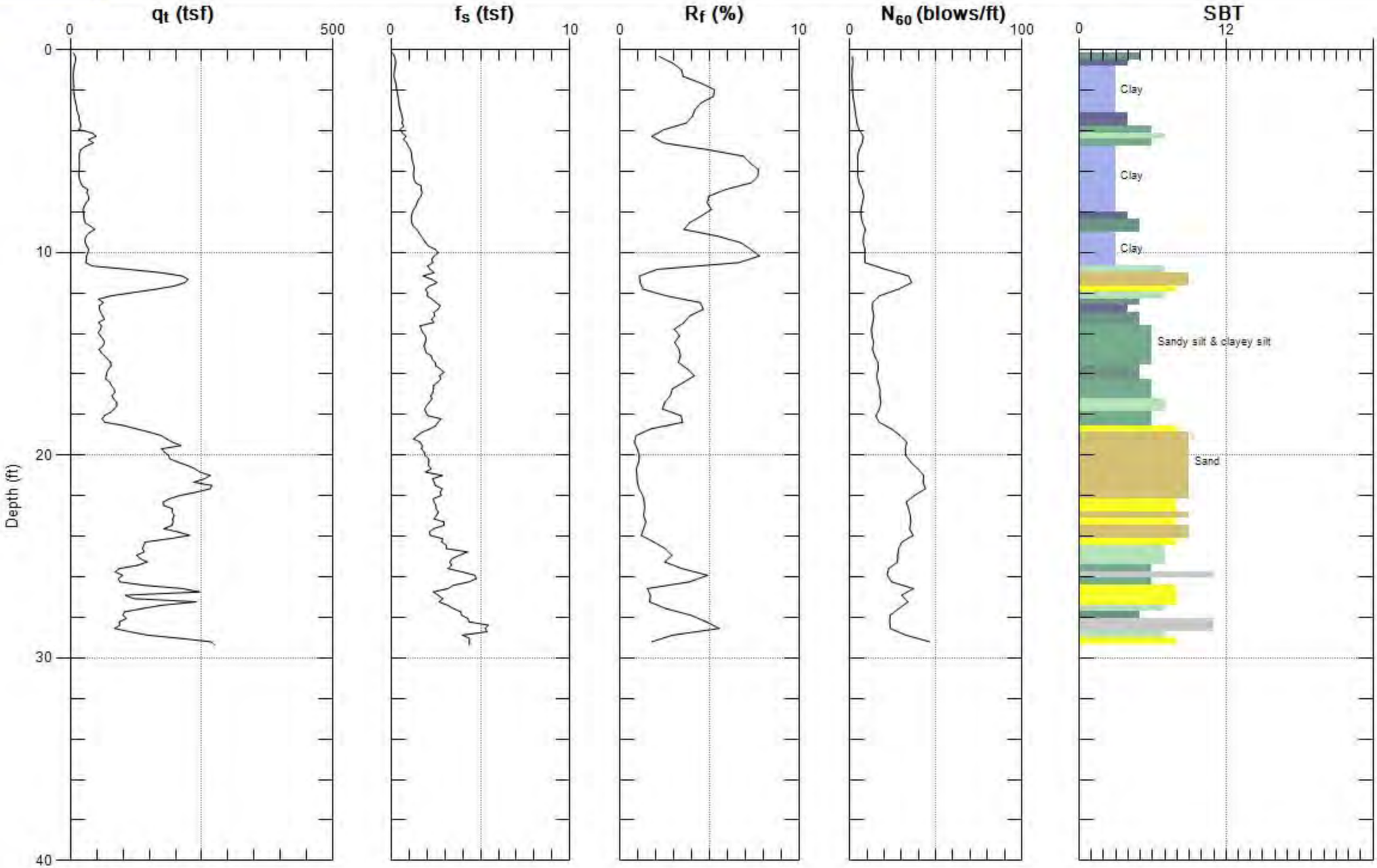

Max. Depth: 10.335 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

# M2 CONSULTANTS

Site: OCSD
Sounding: CPT-4

Engineer: A.JOHNS
Date: 2/20/2012 11:11



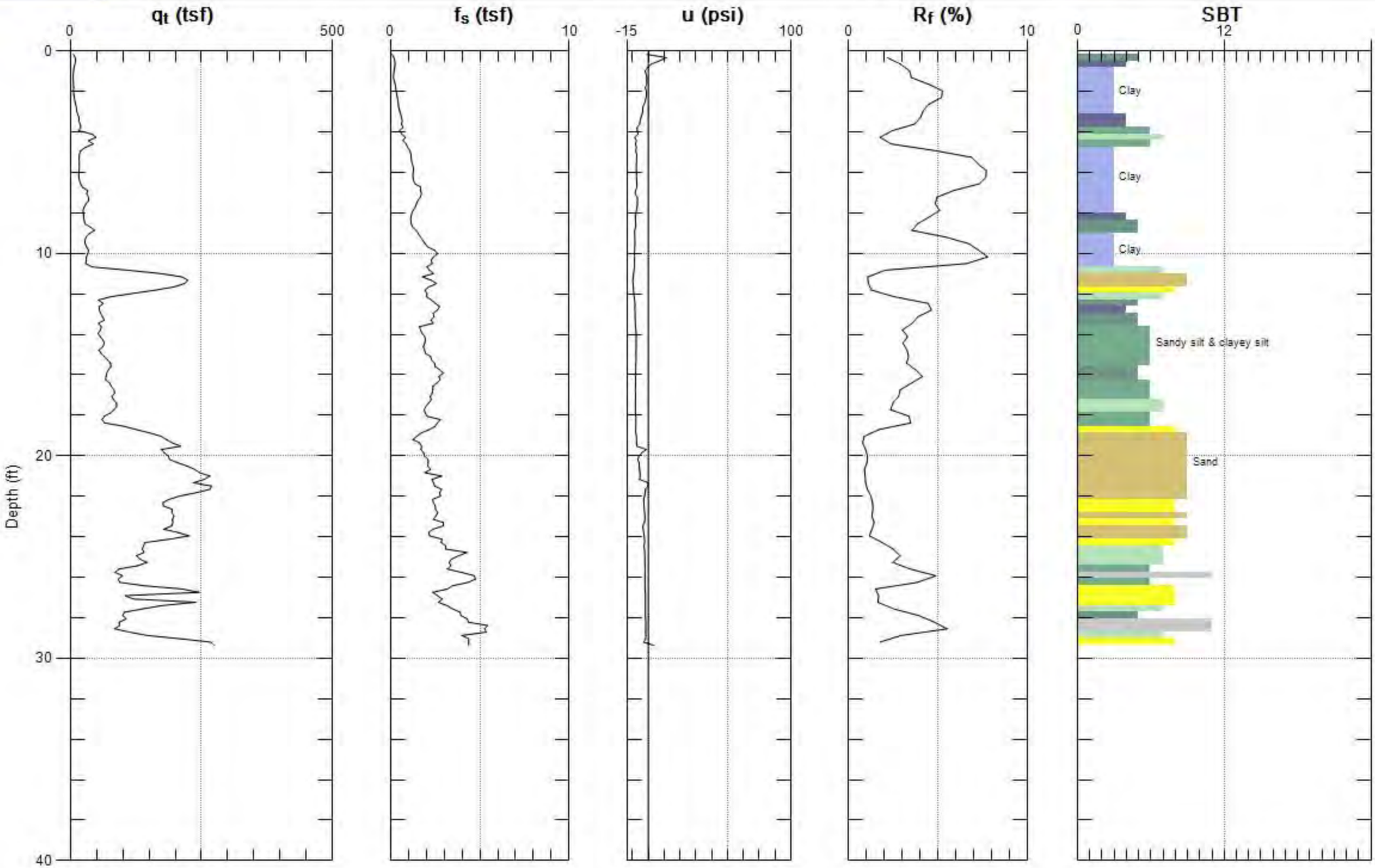

Max. Depth: 29.364 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

# M2 CONSULTANTS

Site: OCSD
Sounding: CPT-4

Engineer: A.JOHNS
Date: 2/20/2012 11:11



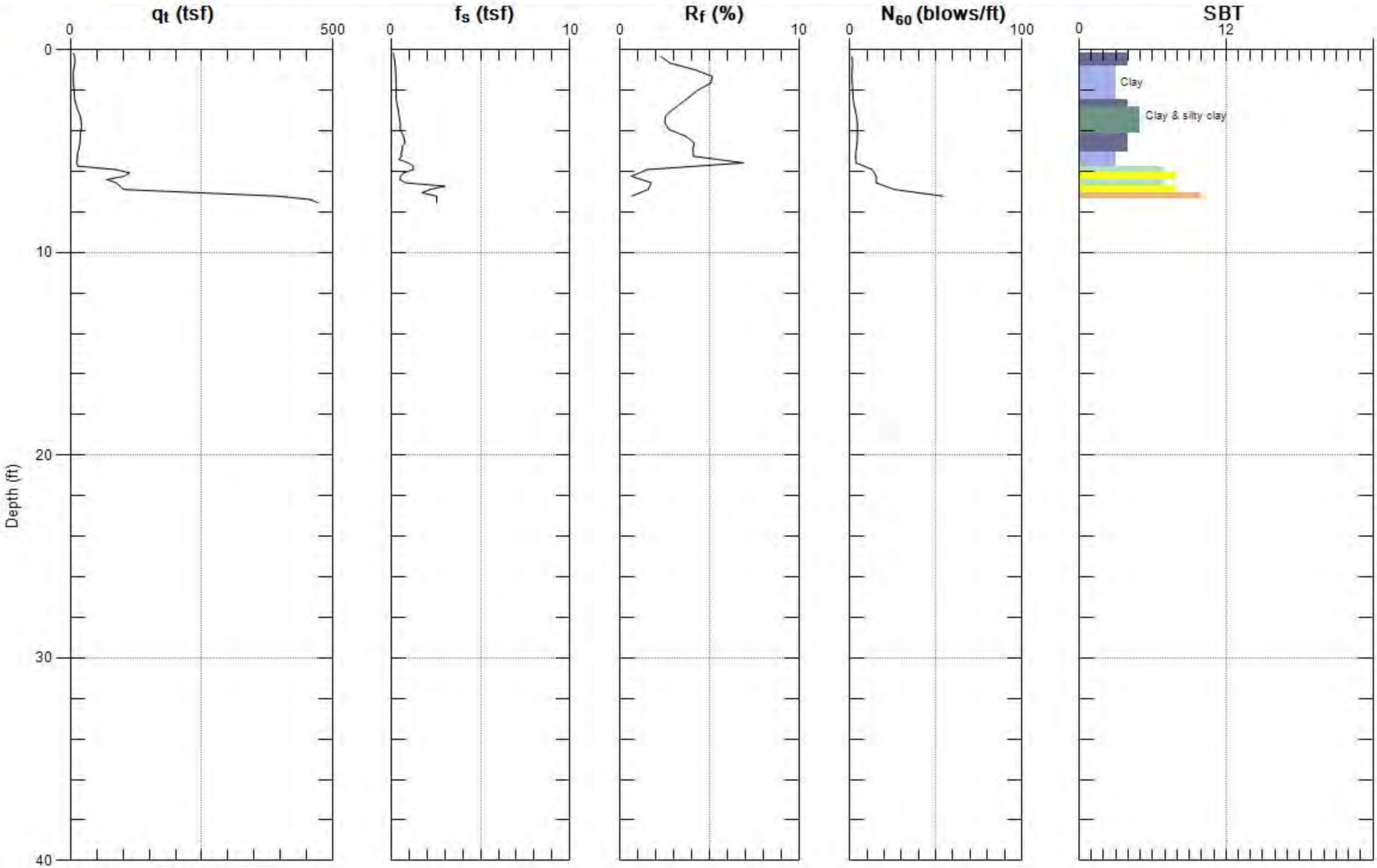

Max. Depth: 29.364 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

# M2 CONSULTANTS

Site: OCSD
Sounding: CPT-5

Engineer: A.JOHNS
Date: 2/20/2012 10:38



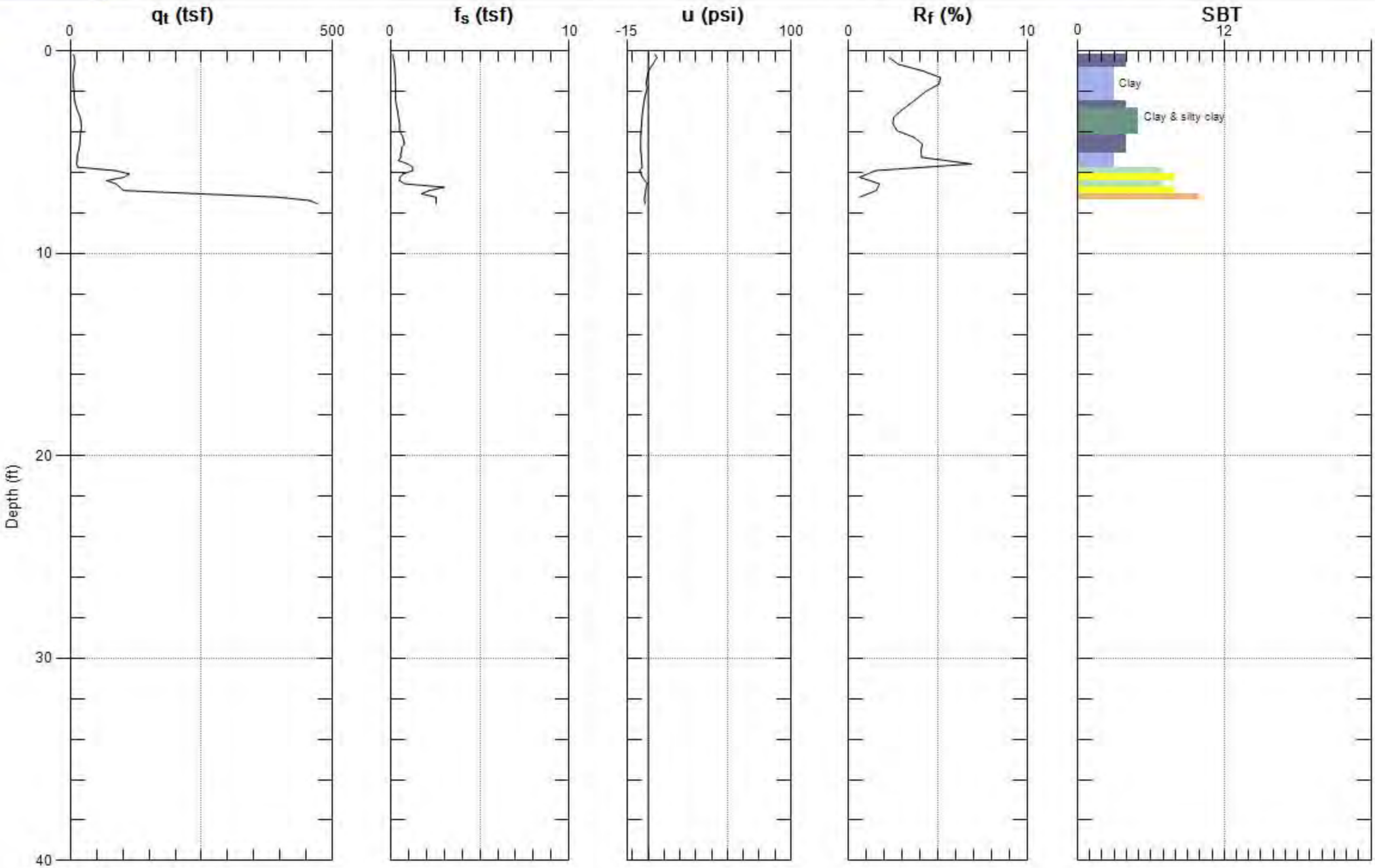

Max. Depth: 7.546 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

# M2 CONSULTANTS

Site: OCSD
Sounding: CPT-5

Engineer: A.JOHNS
Date: 2/20/2012 10:38



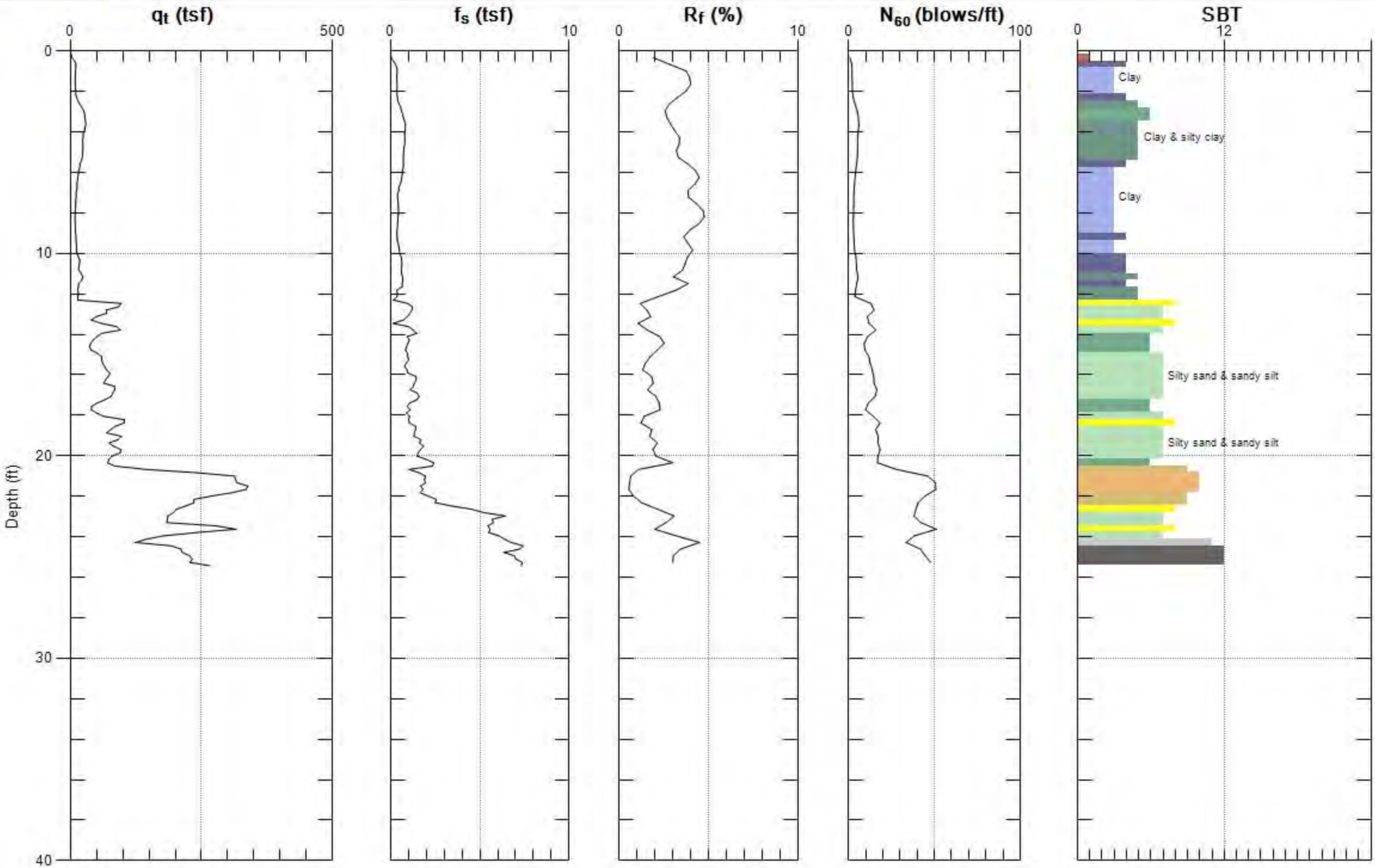

Max. Depth: 7.546 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

# M2 CONSULTANTS

Site: OCSD
Sounding: CPT-6

Engineer: A.JOHNS
Date: 2/20/2012 09:24



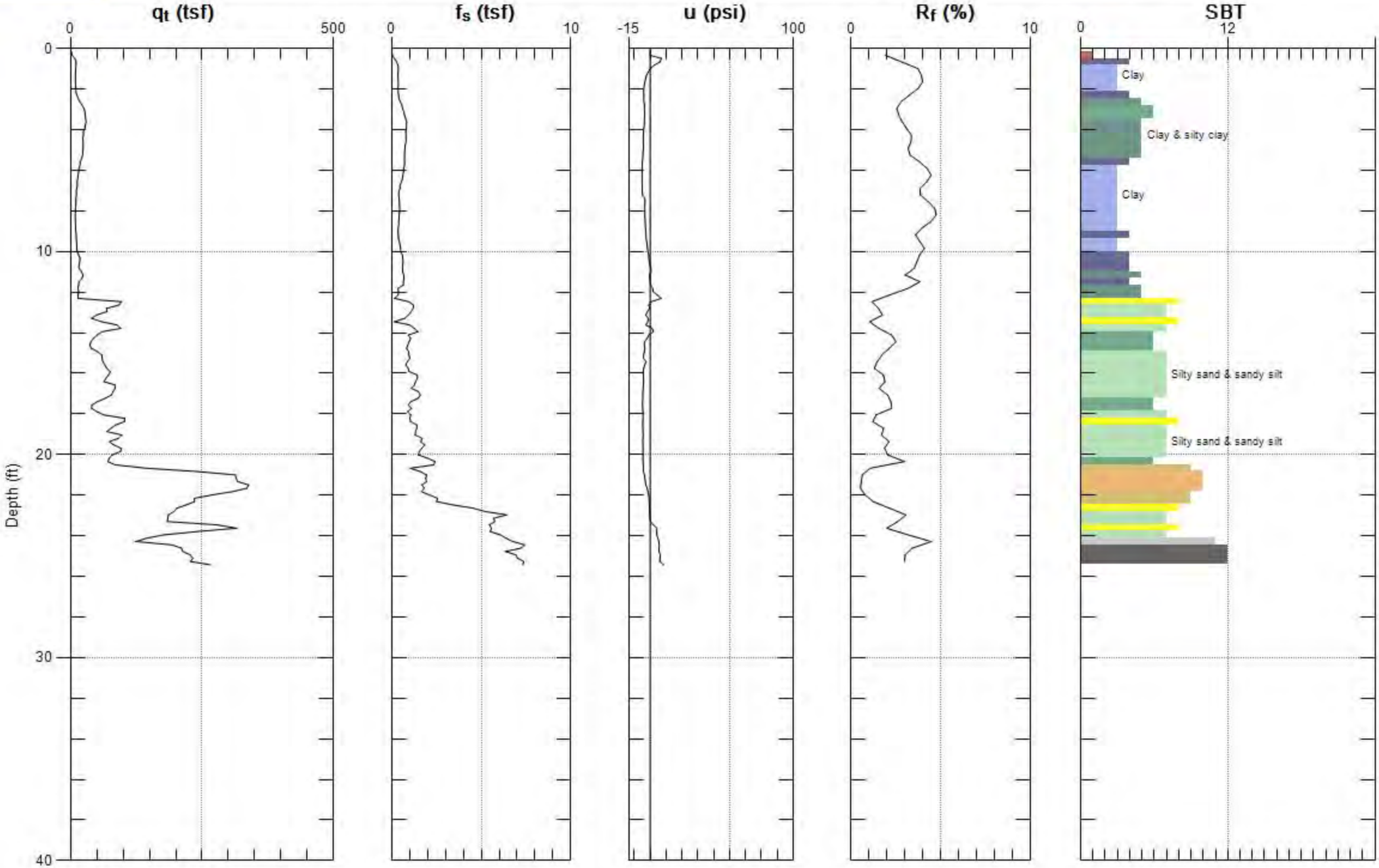

Max. Depth: 25.427 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

# M2 CONSULTANTS

Site: OCSD
Sounding: CPT-6

Engineer: A.JOHNS
Date: 2/20/2012 09:24




Max. Depth: 25.427 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

# Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*. The soundings were conducted using a 20 ton capacity cone with a tip area of 15 $cm^2$ and a friction sleeve area of 225 $cm^2$. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80.

The cone takes measurements of cone bearing ($q_c$), sleeve friction ($f_s$) and penetration pore water pressure ($u_2$) at 5-cm intervals during penetration to provide a nearly continuous log. CPT data reduction and interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored on disk for further analysis and reference. All CPT soundings are performed in accordance with revised (2007) ASTM standards (D 5778-07).

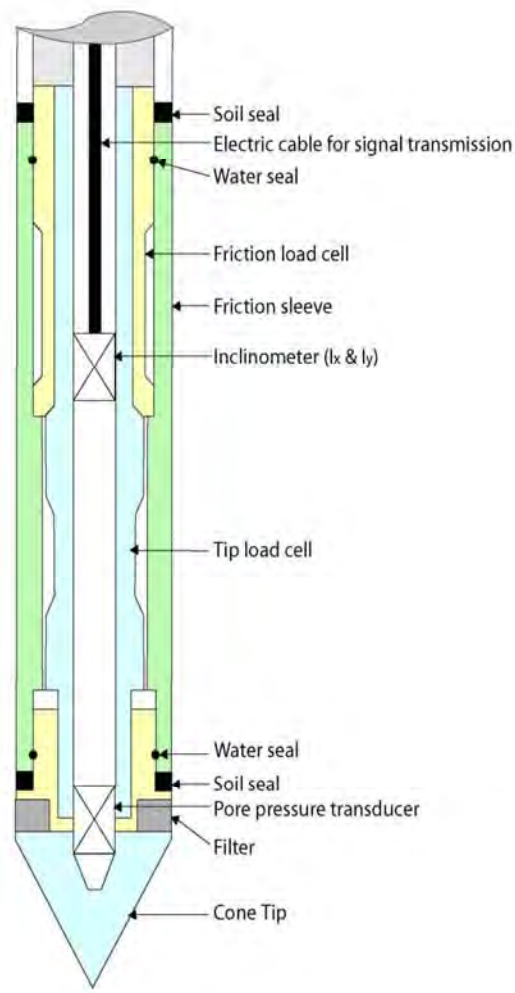



*Figure CPT*

The cone also contains a porous filter element located directly behind the cone tip ($u_2$). It consists of porous plastic and is 5.0mm thick. The filter element is used to obtain penetration pore pressure as the cone is advanced as well as Pore Pressure Dissipation Tests (*PPDT's*) during appropriate pauses in penetration. It should be noted that prior to penetration, the element is fully saturated with oil under vacuum pressure to ensure accurate and fast dissipation.

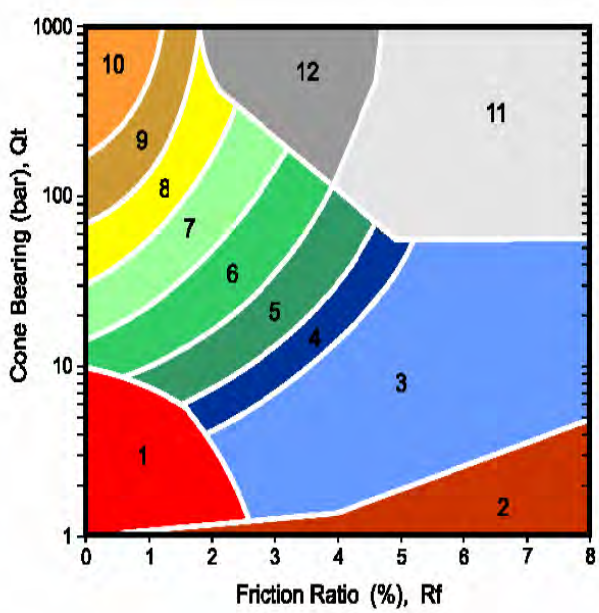
The cone has the following accuracy:
1 tsf for $q_c$, 0.02 tsf for $f_s$ and 0.5 psi for $u_2$. In soft clays, a lower capacity cone should be used for improved accuracy.

When the soundings are complete, the test holes are grouted. The grouting procedures generally consist of pushing a hollow tremie pipe with a "knock out" plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

## CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test (CPT) data collected from your site are presented in graphical form in the attached report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the nonnormalized charts of Robertson et al (1986). For CPT soundings extending greater than 50 feet, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. do not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and do not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user. A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on $q_t$, $f_s$, and $u_2$. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.




*(After Robertson, et al., 1986)*

| ZONE | | SBT |
|---|---|---|
| 1 | | Sensitive, fine grained |
| 2 | | Organic materials |
| 3 | | Clay |
| 4 | | Silty clay to clay |
| 5 | | Clayey silt to silty clay |
| 6 | | Sandy silt to clayey silt |
| 7 | | Silty sand to sandy silt |
| 8 | | Sand to silty sand |
| 9 | | Sand |
| 10 | | Gravely sand to sand |
| 11 | | Very stiff fine grained* |
| 12 | | Sand to clayey sand* |

*over consolidated or cemented

# CONE PENETRATION TEST INTERPRETATION

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

**Input**

1. Units for display (Imperial or metric) (atm. pressure, pa = 0.96 tsf or 0.1 MPa)
2. Depth interval to average results,( ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
3. Elevation of ground surface (ft or m)
4. Depth to water table, zw (ft or m) – input required
5. Net area ratio for cone, a (default to 0.80)
6. Relative Density constant, CDr (default to 350)
7. Young's modulus number for sands, α (default to 5)
8. Small strain shear modulus number
   a. for sands, SG (default to 180 for SBTn 5, 6, 7)
   b. for clays, CG (default to 50 for SBTn 1, 2, 3 & 4)
9. Undrained shear strength cone factor for clays, Nkt (default to 15)
10. Over Consolidation ratio number, kocr (default to 0.3)
11. Unit weight of water, (default to γw = 62.4 lb/ft3 or 9.81 kN/m3)

**Column**

1. Depth, z, (m) – CPT data is collected in meters
2. Depth (ft)
3. Cone resistance, qc (tsf or MPa)
4. Sleeve friction, fs (tsf or MPa)
5. Penetration pore pressure, u (psi or MPa), measured behind the cone (i.e. u2)
6. Other – any additional data, if collected, e.g. electrical resistivity or UVIF
7. Total cone resistance, qt (tsf or MPa) qt = qc + u (1-a)
8. Friction Ratio, $R_f$ (%) $R_f = (f_s/q_t) \times 100\%$

9. Soil Behavior Type (non-normalized), SBT see note
10. Unit weight, $\gamma$ (pcf or kN/m$^3$) based on SBT, see note
11. Total overburden stress, $\sigma_v$ (tsf) $\sigma_{vo} = \gamma z$
12. Insitu pore pressure, $u_o$ (tsf) $u_o = \gamma_w (z - z_w)$
13. Effective overburden stress, $\sigma'_{vo}$ (tsf ) $\sigma'_{vo} = \sigma_{vo} - u_o$
14. Normalized cone resistance, $Q_{t1}$ $Q_{t1} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15. Normalized friction ratio, $F_r$ (%) $F_r = f_s / (q_t - \sigma_{vo})$ x 100%
16. Normalized Pore Pressure ratio, $B_q$ $B_q = u - u_o / (q_t - \sigma_{vo})$
17. Soil Behavior Type (normalized), $SBT_n$ see note
18. $SBT_n$ Index, $I_c$ see note
19. Normalized Cone resistance, $Q_{tn}$ (n varies with Ic) see note
20. Estimated permeability, $k_{SBT}$ (cm/sec or ft/sec) see note
21. Equivalent SPT $N_{60}$, blows/ft see note
22. Equivalent SPT $(N_1)_{60}$ blows/ft see note
23. Estimated Relative Density, $D_r$, (%) see note
24. Estimated Friction Angle, $\varphi'$, (degrees) see note
25. Estimated Young's modulus, $E_s$ (tsf) see note

26 Estimated small strain Shear modulus, Go (tsf) see note

27 Estimated Undrained shear strength, $s_u$ (tsf) see note

28 Estimated Undrained strength ratio $s_u/\sigma_v'$

29 Estimated Over Consolidation ratio, OCR see note

**Notes:**

1. Soil Behavior Type (non-normalized), SBT Lunne et al. (1997)
   listed below
2. Unit weight, $\gamma$ either constant at 119 pcf or based on Non-normalized SBT
   (Lunne et al., 1997 and table below)
3. Soil Behavior Type (Normalized), $SBT_n$ Lunne et al. (1997)
4. $SBT_n$ Index, $I_c$ $I_c = ((3.47 - \log Q_{t1})^2 + (\log F_r + 1.22)^2)^{0.5}$
5. Normalized Cone resistance, $Q_{tn}$ (n varies with Ic)
   $Q_{tn} = ((q_t - \sigma_{vo})/pa)\ (pa/(\sigma'_{vo})^n$ and recalculate $I_c$, then iterate:
   When $I_c < 1.64$, n = 0.5 (clean sand)

When $I_c > 3.30$, $n = 1.0$ (clays)

When $1.64 < I_c < 3.30$, $n = (I_c - 1.64)0.3 + 0.5$

Iterate until the change in n, $\Delta n < 0.01$

# APPENDIX D

# SITE 3C-2

# SEISMIC REFRACTION STUDY

NORCAL GEOPHYSICAL CONSULTANTS, INC.

January 23, 2013

Mr. Alastair Johns
M2 Consultants, Inc.
638 19th Street, Unit 18
San Francisco, CA 94104

Subject: Seismic Refraction Survey
Proposed Pond 3C-2
OCSD Wastewater Storage and Reclamation Project
Occidental, Sonoma County, CA
NORCAL Job # 12-1053.01

Dear Mr. Johns:

This report presents the findings of a seismic refraction (SR) survey performed by NORCAL Geophysical Consultants, Inc. on a parcel of land located in Occidental, CA. The survey was performed on October 17, 2012 by NORCAL Professional Geophysicist Donald J. Kirker and geophysical technician Travis Black. Logistical support was provided by Alastair Johns of M2 Consultants, Inc. (M2).

## 1.0 SITE DESCRIPTION

The seismic refraction survey for the proposed Pond 3C-2 was conducted on a parcel of land presently being used for grazing cattle. It is located on top of a small knoll near the intersection of Occidental Road and Facendini Lane near Occidental, CA. The terrain is characterized by gently rolling hills covered with grass and deciduous and coniferous trees. There are localized outcrops of volcanic tuff evident in the general area outside of the proposed Pond 3C-2 footprint.

The geophysical survey, as specified by M2, was conducted along two perpendicular lines positioned on top of the knoll. They are designated as Lines 1 and 2 on Plate 1, and correspond with the proposed location of the pond. Surface elevations range from 870- to 892-ft above mean sea level (msl).

Recently, M2 drilled several soil borings within this area, as shown on Plate 1. Three of these borings, designated as B-5, B-6, and B-7, are located in the center of the pond and in close proximity to Lines 1 and 2. Borehole logs from these borings indicate a 24- to 33-ft thick sequence of sandstone conglomerate and gravel over bedrock. Sampling blow-count data within the conglomerate shows that it is highly variable. The bedrock consists of alternating layers of meta-shale, shale, and greywacke, and was encountered at depths of 24-to 33-ft bgs. According to the sampling blow-count data, the character of the bedrock is also highly variable. The meta-shale is very strong and only slightly weathered; the graywacke and shale is completely to moderately weathered.

321A BLODGETT STREET • COTATI, CA 94931 • TELEPHONE (707) 796-7170 • FAX (707) 796-7175
www.norcalgeophysical.com

## 2.0 PURPOSE

The purpose of the SR survey was to obtain seismic refraction data within the upper 50-ft to aid in determining the thickness of overburden and the depth and excavation characteristics (rippability) of the bedrock. We understand that M2 will use this information as part of their geotechnical evaluation for the construction of Pond 3C-2.

## 3.0 METHODOLOGY

The SR method is used to determine the compressional velocity of subsurface materials. The seismic velocity of fill, sediments, and rock are dependent on physical properties such as compaction, density, hardness, and induration. However, other factors such as bedding, fracturing, and saturation also affect seismic velocity. Typically, low velocities are indicative of loose soil, poorly compacted fill material, poorly to semi-consolidated sediments, and deeply weathered and highly fractured rock. Moderate velocities are usually indicative of dense and highly compacted sediments and fill, and/or moderately weathered and moderately fractured rock. High velocities are indicative of slightly weathered to unweathered rock with little fracturing. It should be noted that apparent velocities can be affected by the orientation of bedding planes and fracture patterns with respect to the direction of the seismic profile. Apparent velocities of rock are typically slower when measured along lines oriented perpendicular to bedding planes of steeply dipping rock, than those measured along lines oriented parallel. A more detailed description of the SR methodology is provided in Appendix A.

## 4.0 FIELD INVESTIGATIONS

We obtained seismic refraction data along two profiles, as determined by M2. They are designated as Lines 1 and 2 on Plate 1 and measure 510- and 350-ft long, respectively. Line 1 consisted of three overlapping geophone arrays (spreads); Line 2 consisted of two. For this survey a spread consisted of 5-shot points and 24-geophones distributed in a collinear array. Along both lines, the geophones were coupled to the ground surface at 10 foot intervals. For each spread, two of the shot points were located 5- to 10-ft beyond the end geophones and the remaining shot points were evenly spaced within the spread.

The SR data were recorded using a Geometrics ***Geode***, 24-bit digital seismic recording system and Oyo Geospace digital-grade geophones with a natural frequency of 10-Hz. We produced seismic energy at each shot point by striking an aluminum plate, placed on the ground surface, with a 16-pound sledge hammer. An accelerometer attached to the hammer transmitted a triggering pulse to the seismograph each time the plate was struck. The resulting travel time data were recorded on a seismograph and processed to generate seismic velocity cross-sections.

We used a Trimble global positioning system (GPS) with sub-foot accuracy to measure the geographical coordinates of the shot points along each line. These positions were differentially corrected and added to a site diagram provided by M2 to create Plate 1. A more detailed description of data acquisition and analysis procedures are also provided in Appendix A.

## 5.0 RESULTS

The results of the seismic refraction survey are illustrated by the seismic velocity profiles shown on Plate 2. On each profile the vertical axis represents elevation (above mean sea level) and the horizontal axis represents station distance in feet. The profiles depict the ground surface and gradational color shading that represents seismic velocities according to the color scale shown at the bottom of the plate.

### 5.1 SEISMIC REFRACTION PROFILES

The profiles shown on Plate 2 indicate seismic velocities of 1000- to over 9,000-ft/s to depths of approximately 60-ft. This is manifested by continuous velocity contours that generally parallel the ground surface. These contours show a surface layer of low velocities (1000- to 2000-ft/s) that extend 5- to 20 -ft below ground surface (bgs). Beneath this low velocity zone (brown shading) there is a relatively thin (9- to 18-ft) zone where contours are closely spaced and velocities increase rapidly from 2000- to 7000-ft/s (yellow, green, and blue). This overlies a subsurface that consists primarily of high velocities of 7000- to 9000-ft/s (purple shading) at depths of 15- to 52-ft.

### 5.2 BORING LOG CORRELATION AND INTERPRETATION

On each profile, we have superimposed the respective geologic logs for borings B-5 through B-7. As shown on Plate 1, Borings B-5 and B-7 are located along Line 1; boring B-6 is located along Line 2. Each log is shown as a column with specific patterns representing conglomerate and greywacke/shale bedrock. B-6 also shows patterns for the thin layer of gravel defined in that boring only. This boring log information provides a means for correlating lithologic units with specific seismic velocity ranges.

Based on the information from the geologic logs and the velocity trends described above, we interpret velocities of 1000- to 7,000 ft/s as representing the sandstone conglomerate. We believe that the wide range of velocities within the conglomerate is the result of an increase in density with depth. This is based on the significant increase in sampling blow-counts evident on each borehole log at depths (15- to 20-ft) where velocities increase rapidly on each profile. Velocities of over 7,000-ft/s are characteristic of the slightly weathered meta-shale bedrock. Based on the highly weathered characterization of the greywacke and shale layers beneath the meta-shale, as defined in the borehole logs, the velocities of this material should be less than indicated on the seismic refraction profiles. If this is the case these materials cannot be accurately determined since velocity inversions (high velocity material over low velocity material) cannot be defined by the seismic refraction method.

## 6.0 EXCAVATION CHARACTERISTICS (Rippability)

Seismic velocity charts relating seismic velocity and excavation characteristics have been developed from field tests by others. These charts list the seismic velocity of various types of rock and their relative ease of excavation using different types of rippers. Caterpillar Tractor Company publishes a performance manual that lists ripper performance charts for the D8L, D9L, and D11L tractors. The following information in Table A was obtained from a performance chart for a D9L Ripper:

Table A: D9L Ripper Performance Chart

| PERFORMANCE | ROCK TYPE | VELOCITY RANGE (ft/s) |
|---|---|---|
| Rippable | Sedimentary | < 6,400 to 7,800 |
| | Igneous | < 6,700 to 7,600 |
| | Metamorphic | < 7,200 to 7,300 |
| Marginally Rippable | Sedimentary | 6,400 to 9,700 |
| | Igneous | 6,700 to 8,600 |
| | Metamorphic | 7,200 to 9,200 |
| Non-rippable | Sedimentary | > 8,600 to 9,700 |
| | Igneous | > 8,000 to 8,700 |
| | Metamorphic | > 9,000 to 9,200 |

According to the D9L Ripper Performance chart above, the highest velocities defined along Lines 1 and 2 (7,000- to 9,000-ft/s) are consistent with material that is marginally rippable to non-rippable. However, this information should only be used as a general guide, as there are many other factors that should also be considered. These factors include rock jointing and fracture patterns, the experience of the equipment operator, and the equipment and excavation methods selected. Also, the computed velocities measured along each profile are an average for each layer. Therefore, there may be localized zones within each layer where the velocities may be higher or lower than indicated. Since the accuracy of our findings is subject to these limitations, it should be noted that subsurface conditions may vary slightly from those depicted in the final results.

## 7.0 STANDARD OF CARE

The scope of NORCAL's services for this project consisted of using geophysical methods to characterize the subsurface. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the standard of care ordinarily exercised by members of the profession currently

employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

We appreciate having the opportunity to provide you with this information.

Respectfully,

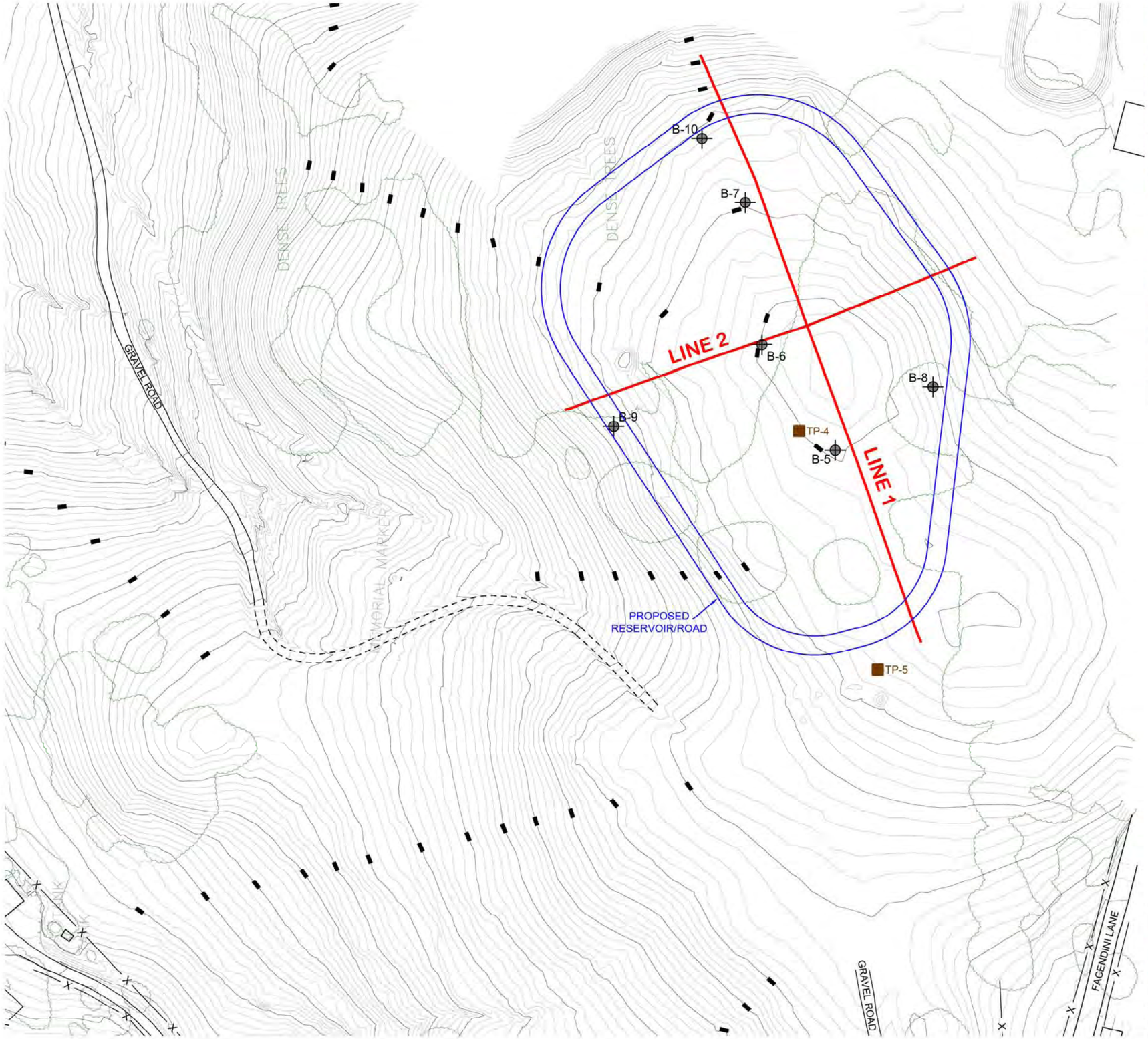
NORCAL Geophysical Consultants, Inc.

Donald J. Kirker

Donald J. Kirker
Professional Geophysicist, PGp-997

DJK/tt

Enclosures: Plates 1 and 2
Appendix A Seismic Refraction Survey



VICINITY MAP

SCALE

0 40 80 160

(1 inch = 80 feet)

## LEGEND

| | |
|---|---|
| | SEISMIC REFRACTION LINE |
| B-5 | BORING LOCATION |
| TP-5 | FORMER TRENCH LOCATION |

NOTE: BASE MAP PROVIDED BY BRELJE & RACE

NORCAL

GEOPHYSICAL SURVEY MAP
SEISMIC REFRACTION SURVEY
PROPOSED POND 3C-2 • OCSD WASTEWATER
STORAGE AND RECLAMATION PROJECT

LOCATION: SONOMA COUNTY, CALIFORNIA

CLIENT: M2 CONSULTANTS, INC.

JOB #: 12-1053.01

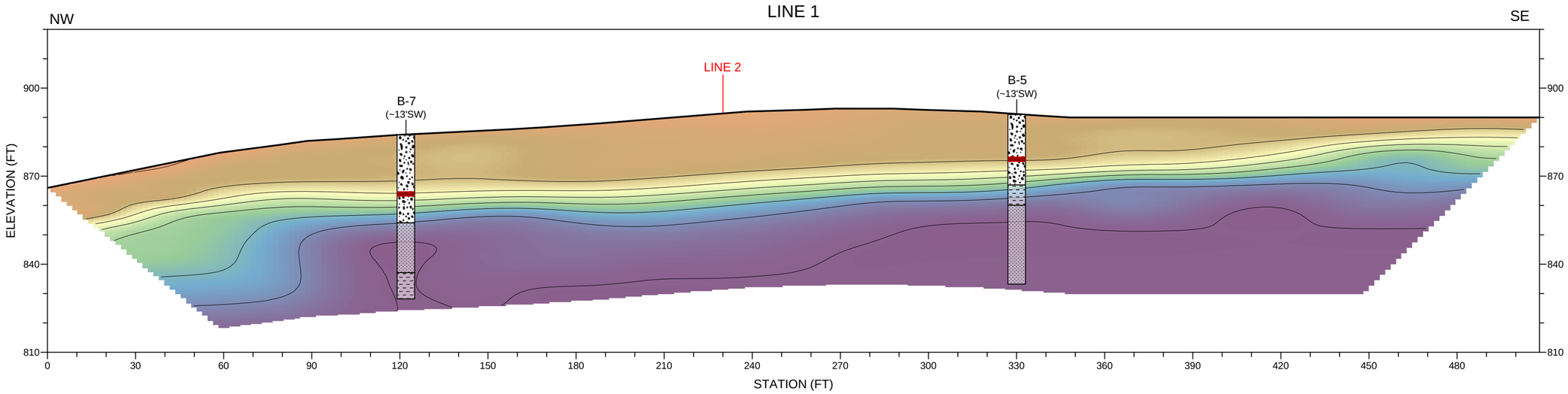
NORCAL GEOPHYSICAL CONSULTANTS INC.

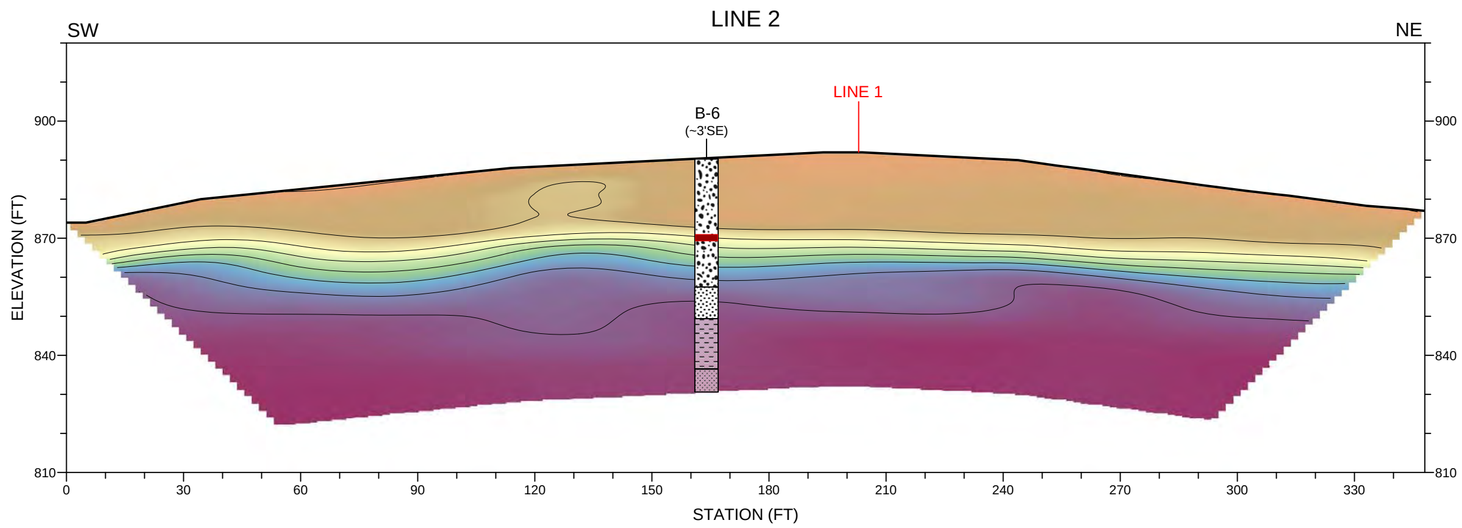
DATE: DEC. 2012

DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 1




| BORING LOGS | |
|---|---|
| | CONGLOMERATE |
| | GRAVEL |
| | SHALE |
| | GRAYWACKE |
| | SIGNIFICANT INCREASE IN SAMPLING BLOW COUNTS |

VELOCITY (FT/SEC)

1000 2000 3000 4000 5000 6000 7000 8000 9000

SCALE

0 30 60

(1 inch = 30 feet)

NORCAL

GEOPHYSICAL SURVEY MAP
SEISMIC REFRACTION SURVEY
PROPOSED POND 3C-2 ● OCSD WASTEWATER
STORAGE AND RECLAMATION PROJECT

LOCATION: SONOMA COUNTY, CALIFORNIA

CLIENT: M2 CONSULTANTS, INC.

JOB #: 12-1053.01

NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE: DEC. 2012

DRAWN BY: G.RANDALL

APPROVED BY: DJK

PLATE 2

# APPENDIX E

# TABLE 1

# GEOLOGIC HAZARDS & CONSTRUCTION CONSIDERATIONS

| TABLE 1 - GEOLOGIC HAZARDS & CONSTRUCTION CONSIDERATIONS | | | | |
|---|---|---|---|---|
| **Geologic Hazard** | **Loades South** | **Loades North[1]** | **Briggs** | **Colella** |
| Surface Fault Rupture | No active faults traverse parcel | No active faults traverse parcel | No active faults traverse parcel | No active faults traverse parcel |
| Landslides | No landslides-suspected | 1. Large mapped & partially evacuated but unremediated old complex landslide present on southern portion of parcel immediately north of Loades residence. Slip plane potentially present at depth<br>2. Two more recent (historic?) small and likely fairly shallow rotational landslides present on west bank of existing Loades Pond | No landslide-suspected | Large mapped & partially evacuated but unremediated old complex landslide on about 60% of parcel. Slip plane potentially present at depth |
| Slumping | No slumping suspected | Minor near-surface slumping features present on some over-steepened slopes. Deeper slumping features present within large mapped landslide mass noted above | No slumping suspected | Some slumping present within large mapped landslide mass noted above |
| Slope Creep | Near surface creep present on over steepened slopes | Near surface creep present on over steepened slopes | Minor creep present at steeper north end of the parcel | Creep present at steeper north end of parcel |
| Potentially Expansive Soils[2] | Sheared serpentinite weathered to expansive fat clay materials at numerous outcrops / cuts / erosional areas. Distribution of potentially expansive soils is highly variable | Sheared serpentinite weathered to expansive fat clay materials at several outcrops / cuts / erosional areas but not as widespread as in Loades South parcel. Distribution of potentially expansive soils is highly variable | None encountered | Sheared serpentinite present in old landslide deposits and at head of slide, but doesn't appear as potentially expansive as in Loades South & North parcels |
| Seismic Shaking | Strong ground shaking should be expected throughout entire parcel during seismic events on nearby active faults, especially large earthquakes on the nearby San Andreas & Rogers Creek fault segments | Strong ground shaking should be expected throughout entire parcel during seismic events on nearby active faults, especially large earthquakes on the nearby San Andreas & Rogers Creek fault segments | Strong ground shaking should be expected throughout entire parcel during seismic events on nearby active faults, especially large earthquakes on the nearby San Andreas & Rogers Creek fault segments | Strong ground shaking should be expected throughout entire parcel during seismic events on nearby active faults, especially large earthquakes on the nearby San Andreas & Rogers Creek fault segments |
| Liquefaction & Lateral Spreading[3] | Near surface bedrock generally present throughout parcel therefore the potential for liquefaction and lateral spreading is low. | The flatland's portion of the the Loades North parcels on both the east and west sides is underlain by mapped alluvial deposits with apparently high seasonal groundwater, therefore site specific liquefaction/lateral spreading analysis of both east and west flatland areas utilizing subsurface data will be required | The flatter southern portion is underlain by mapped alluvial deposits, therefore the potential for liquefaction and lateral spreading should be evaluated during a site specific subsurface investigation | The flatter southern portion is underlain by mapped alluvial and landslide deposits, therefore the potential for liquefaction and lateral spreading should be evaluated during a site specific subsurface investigation |

1: Includes existing Loades Pond
2: Potentially expansive soils identified by field classification only. No laboratory test data available therefore assessment is subjective and preliminary
3 Assessment of liquefaction & lateral spreading based on generalized available published data, site mapping and conservative assumptions only. No quantitative subsurface data or laboratory test results available therefore assessment is subjective and may well change

| TABLE 1 - GEOLOGIC HAZARDS & CONSTRUCTION CONSIDERATIONS | | | | |
|---|---|---|---|---|
| **Geologic Hazard** | **Loades South** | **Loades North** | **Briggs** | **Colella** |
| Dynamic Compaction[4] | Near surface bedrock present throughout parcel therefore the potential for dynamic compaction is low | The flatland's portion of the the Loades North parcels on both the east and west sides is underlain by mapped alluvial deposits with apparently high seasonal groundwater, therefore investigation dynamic compaction potential on both east and west flatland areas utilizing subsurface data will be required | The flatter southern portion is underlain by mapped alluvial deposits, therefore the potential for dynamic compaction potential will be required | The flatter southern portion is underlain by mapped alluvial deposits, therefore the potential for dynamic compaction potential will be required |
| Tsunami, Seiche & Flooding | Not in published FEMA flood zone area | Not in published FEMA flood zone area. Seasonal over-bank flooding of existing creeks along Graton Road appears likely | Not in published FEMA flood zone area. Seasonal over-bank flooding of existing creeks along Graton Road appears likely | Not in published FEMA flood zone area. Seasonal over-bank flooding of existing creeks along Graton Road appears likely |
| Erosional Features & Seeps | Erosional features, primarily free surface water drainage are present | Erosional features, primarily free surface water drainage and erosion from steep the free face of the serpentine outcrop on the west-side of the Loades pond are present | No significant features apparent | No significant features apparent |
| Rippability[5] | The site is underlain by both softer serpentine and harder volcanics. Excavation by tracked dozers is plausible in softer strata. | The potential pond site A is underlain by primarily by alluvial soils, therefore not applicable | The site is underlain by primarily by alluvial soils, therefore not applicable | The site is underlain by primarily by alluvial soils and landslide deposits, therefore not applicable |
| Ground Improvement | Bedrock site therefore not applicable | If liquefaction related geologic hazards are present in the northern flatlands areas then it may be necessary to density upper horizon soils. There are numerous ground improvement techniques that can be evaluated for suitability | If liquefaction related geologic hazards are present in the southern flatlands area then it may be necessary to density upper horizon soils. There are numerous ground improvement techniques that can be evaluated for suitability | If liquefaction related geologic hazards are present in the southern flatlands area then it may be necessary to density upper horizon soils. There are numerous ground improvement techniques that can be evaluated for suitability |
| Engineered Fill Suitability | Bedrock material will likely be variable but with screening, can likely be re-used as engineered fill | The alluvial sediments can likely be re-used as engineered fill after screening and plasticity testing potential | The alluvial sediments can likely be re-used as engineered fill after screening and plasticity testing potential | The alluvial sediments can likely be re-used as engineered fill after screening and plasticity testing potential |

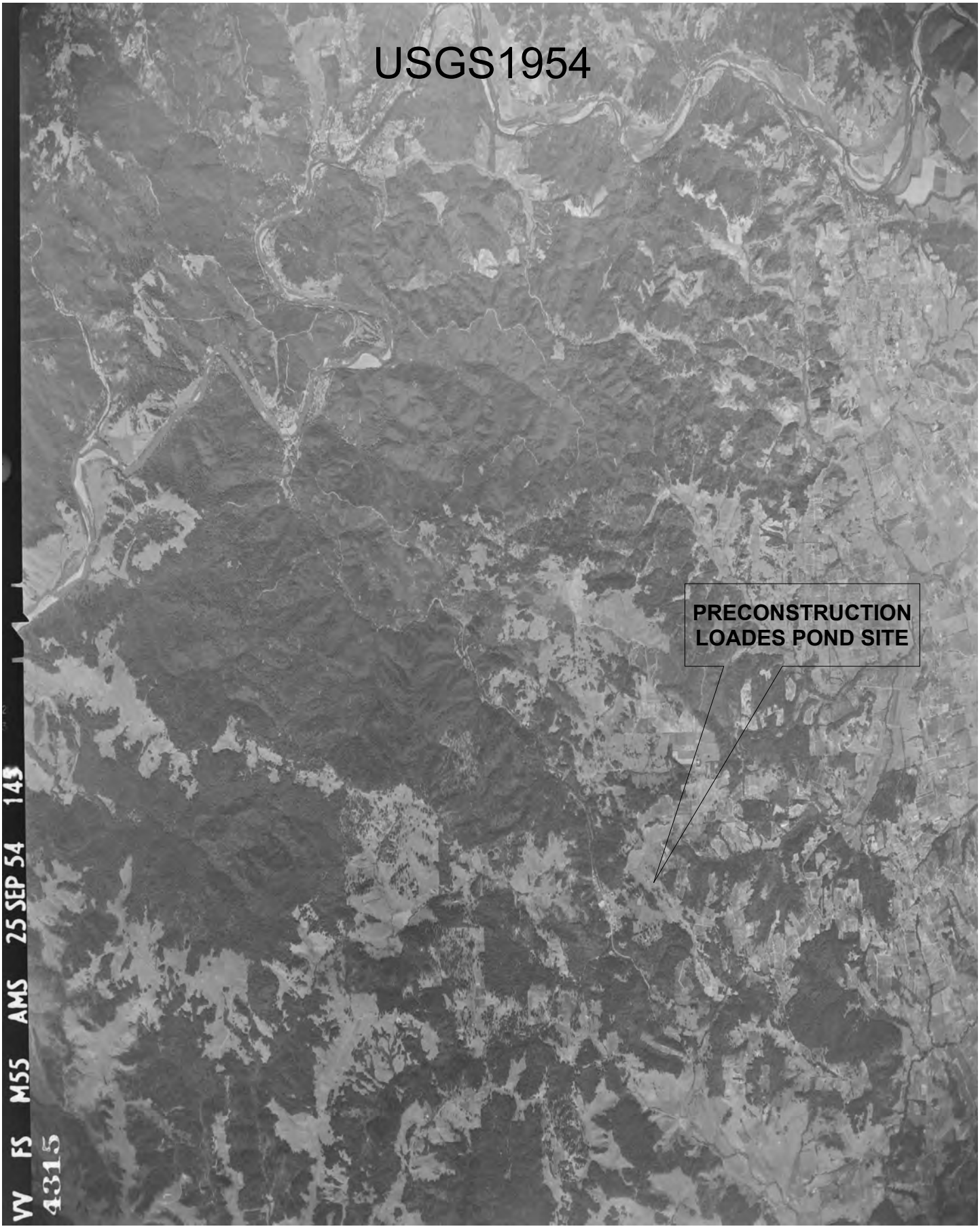
4 Assessment of dynamic compaction based on generalized available published data, site mapping and conservative assumptions only. No quantitative subsurface data or laboratory test results available therefore assessment is subjective and may well change
5 Seismic refraction lines on the Loades South bedrock site would likely be beneficial to quantify near surface (upper 25 feet) existing subsurface bedrock rock densities

# APPENDIX F

# AERIAL PHOTOGRAPHS

10 22 93
1:12000
AV 4252
0210

4-11-80
CIR-SON-13


USGS1954
PRECONSTRUCTION
LOADES POND SITE
VV FS M55 AMS 25 SEP 54 143
4315

# USGS 1952

6-10-52
1-109
GS -UX
PRECONSTRUCTION
LOADES POND SITE
6-10-52
1-108
GS -UX

# APPENDIX G

# SERPENTINITE FORMATION INFORMATION

## SERPENTINE- AND CARBONATE-HOSTED ASBESTOS DEPOSITS
## (MODELS 8d, 18e; Page, 1986; Wrucke and Shride, 1986)

by Chester T. Wrucke

SUMMARY OF RELEVANT GEOLOGIC, GEOENVIRONMENTAL, AND GEOPHYSICAL INFORMATION

Deposit geology

The term asbestos as used in this discussion is a collective term applied by mineralogists to certain fibrous minerals and not to other mineral species that are called asbestos in common parlance. The particular asbestos mineral that is the focus of this discussion is chrysotile.

Serpentine-hosted asbestos: Chrysotile (white asbestos) forms veins in serpentinized ultramafic rocks. Protoliths include ophiolite and stratiform complexes. The veins commonly fill fractures developed in shear zones, notably near changes in rock competency, as near contacts between serpentinite bodies and igneous masses emplaced into serpentinite. Minor asbestos veins are present in unaltered ultramafic rocks adjacent to serpentinite.

Carbonate-hosted asbestos: Veins of cross-fiber chrysotile asbestos are in tabular masses of serpentinite that replaced metalimestone during contact metamorphism related to intrusion of diabase sills, sheets, and dikes. The metalimestone developed from a cherty dolostone protolith early in the metamorphic event. Asbestos veins filled fractures incrementally during dilation caused by the diabase intrusion.

Examples

Serpentine-hosted: Canada- Thetford Mines, Black Lake, and Asbestos-Shipton areas, Quebec; Cassair, British Colombia; White Bay area, Newfoundland (Riordon, 1957; Virta and Mann, 1994). United States- Belvidere Mountain, Vt. (an extension of the Quebec deposits) (Chidester and others, 1978); Calaveras and San Benito Counties, Calif. (Rice, 1966; Virta and Mann, 1994; Coleman, in press). Africa and Europe- Bazhenovo district, central Ural Mountains, Russia (Virta and Mann, 1994); Balangero, Italy (Virta and Mann, 1994); Barberton area, South Africa (Sinclair, 1959; Anhaeusser, 1986); Troodos Complex, Cyprus (Virta and Mann, 1994); Shabani, Zimbabwe (Anhaeusser, 1986; Virta and Mann, 1994).

Carbonate-hosted: Gila County, Ariz. (Shride, 1969; 1973); Barberton-Caroline district and near Kanye, South Africa (Sinclair, 1959; Anhaeusser, 1986); Laiynan district, Hobei Province, China (Sinclair, 1959).

Spatially and (or) genetically related deposit types

Related deposits include metadolomite-hosted talc and talc associated with serpentinized ultramafic rocks (Brown, 1973; Chidester and others, 1978); no deposit models have been developed for these deposit types.

Potential environmental considerations

(1) Most natural exposures of asbestos-bearing rock, particularly serpentinite derived from ultramafic rocks, are readily eroded by natural agents and the activities of man because most serpentinite is composed of weak, highly fractured rock; however, some serpentinite bodies are highly resistant to erosion.

(2) Sedimentary deposits and debris slides derived from asbestos-bearing rocks provide asbestos for redistribution by water and wind.

(3) Vehicles driven across serpentinite and mine waste can dislodge asbestos, adding it to dust or making it readily available to surface drainage; roads also produce channels that aid run-off. The surface area of roads in the southern half of the chrysotile-bearing New Idria serpentinite in San Benito County, Calif., exceeds the area disturbed by the three largest asbestos mines in this area (Woodward-Clyde Associates, 1989).

(4) Waste generated from asbestos mining and milling operations exposes asbestos to erosion by natural agents. The U.S. Environmental Protection Agency (EPA) considers mine waste that contains more than 1 volume percent asbestos hazardous (Derkies, 1985, p. 4-34). The California Air Resources Board considers asbestos contents of mine waste greater than 5 volume percent as a potential toxic hazard (Resolution 91-27, April 1990).

(5) Asbestos fibers can be incorporated by surface drainage in areas of asbestos-bearing rocks and mines. In central California, water in the California Aqueduct System contains asbestos (Kanarek and others, 1980; Coleman, in press). However, the EPA has concluded that ingestion of asbestos fibers poses no significant cancer risk (U.S. Environmental Protection Agency, 1991).

(6) Chrysotile deposits may contain small amounts of fibrous tremolite, which is classifiable by EPA as asbestos and a risk to human health (U.S. Occupational Safety and Health Administration, 1975).

7) Health risks to humans from exposure to small quantities of chrysotile asbestos in the environment are controversial. The controversy results from the EPA assumption that any amount of asbestos is potentially hazardous.

Exploration geophysics

Remote sensing techniques can detect belts of ultramafic rocks, intrusive masses, residual iron oxide minerals, and serpentinite by infra red reflectance, thermal properties, and botanical anomalies (stress and density) in areas of serpentinite-hosted chrysotile, and can identify diabase outcrops in areas of carbonate-hosted chrysotile. Magnetic techniques can be used to identify serpentinite derived from ultramafic rocks because of their high magnetite content. However, the method may not be useful in identification of serpentinite developed from carbonate rocks. This serpentinite may contain magnetite, but the protoliths are carbonate rocks that generally have a low initial iron content, and associated serpentinite bodies are small.

References

Geology: Shride (1969, 1973), Chidester and others, (1978), Ross (1981), Anhaeusser (1986), Virta and Mann (1994).
Environmental geology: Derkies (1985), Coleman (1995).

GEOLOGIC FACTORS THAT INFLUENCE POTENTIAL ENVIRONMENTAL EFFECTS

Deposit size

Deposit size varies from large in serpentine-hosted deposits to small in carbonate-hosted deposits. Serpentine-hosted deposits commonly contain hundreds of million tonnes of ore. In the 1950s the asbestos deposit at the Jeffrey Mine in Quebec had ore reserves as large as 450 million tonnes (Shride, 1973). By the mid 1970s, this mine, which at that time was the largest known asbestos deposit in the world, had the capacity to produce 544,000 tonnes of asbestos fiber a year (Energy, Mines, and Resources Canada, 1976). The mine was an open pit 600 m across in 1983 (Mann, 1983). The Bazhenovo district of Russia may contain larger deposits. This district had the capacity to produce at least 1.36 million tonnes of asbestos annually in the early 1980s (Mann, 1983). Reserves of chrysotile fiber at the asbestos deposit near Copperopolis, Calaveras County, Calif., are reported to be about 1.2 million tonnes (Rice, 1966). The deposit is now closed and is used as an asbestos waste dump. Production of chrysotile from the New Idria serpentinite in San Benito County, Calif. (the largest asbestos deposit in the United States) is now confined to one property, that of the King City Asbestos Corporation (KCAC, Inc.). In 1975 production from this property was 68,000 tonnes (Mumpton and Thompson, 1975). In the past, The New Idria serpentinite was the locale of many mining ventures, including chrysotile asbestos mining. All but one of these asbestos mines have ceased operations, and part of the properties are now EPA superfund sites. In 1989, the latest year for which data are available from the U.S. Bureau of Mines, the production of chrysotile from the KCAC Inc. mine and one in Orleans County, Vt. totalled 17,427 tonnes (Virta, 1989). In contrast, Arizona deposits contain a few tens to a few thousand tonnes of chrysotile asbestos (Shride, 1969). Data on chrysotile mining in Arizona (Shride, 1969 and the Yearbooks of the U.S. Bureau of Mines) for the period 1914 to 1982 (when mining ceased) indicate production of about 90,000 tonnes of asbestos fiber.

Total United States asbestos production in 1989 was 17,427 tonnes compared to world production of 4,237,659 tonnes (Virta, 1991). About 98 percent of the asbestos produced in 1990 worldwide was chrysotile (Virta and Mann, 1994).

Host rocks

Serpentine-hosted deposits: These deposits are in massive serpentinite, commonly highly sheared and widely exposed, that has largely replaced the host ultramafic protolith. Associated ultramafic rocks locally host asbestos veins.
Carbonate-hosted deposits: Serpentinite in these deposits has replaced metalimestone or dolostone. In Arizona, the serpentinite bodies commonly are 1 to 3 m thick, are present at a few stratigraphic intervals in the host metalimestone section, and are structureless, except for asbestos veins.

Surrounding geologic terrane

Serpentine-hosted deposits: Most of these deposits have developed in ophiolite complexes, which are composed of oceanic crustal fragments consisting of a basal peridotite (that becomes serpentinized) overlain in sequence by cumulate gabbro, sheeted dikes, and pillow basalt, commonly capped by deep oceanic pelagic strata. Accreted ophiolite commonly is dismembered into structurally complicated fragments. In Quebec and New England, these rocks are associated with metamorphosed sedimentary and igneous rocks, including schist, greenstone, quartzite, and

amphibolite, of early Paleozoic age. The New Idria serpentinite, Calif., which hosts the KCAC asbestos deposit and others, is completely separated from mafic oceanic crust and is surrounded by sedimentary rocks of the Cretaceous Franciscan and Panoche Formations. These rocks are folded with unconformably overlying Tertiary marine sedimentary rocks.
Carbonate-hosted deposits: Deposits of this type in Arizona are in Middle Proterozoic rocks that include siltstone, arkosic arenite, cherty dolostone, argillite, quartz arenite, local basalt, and intrusions of diabase sheets, sills, and dikes.

Wall-rock alteration

Serpentinization of host rocks and subsequent serpentinite alteration are common kinds of wall-rock alteration associated with asbestos. In serpentine-hosted deposits, serpentinite results from hydration of igneous protoliths, commonly harzburgite and dunite, which are unstable in the presence of water at crustal temperatures (Coleman, 1971; Coleman and Jove, 1993). For example, the mineralogical nature of the New Idria serpentinite probably is related to the relative amounts of orthopyroxene and olivine in the primary peridotite. Reactions with water can be written:

1) $3Mg_2Si_2O_6$ (pyroxene) + $4H_2O$ = $2Mg_3Si_2O_5(OH)_4$ (serpentine) + $2SiO_2$ (quartz)
2) $2Mg_2SiO_4$ (olivine) + $3H_2O$ = $Mg_3Si_2O_5(OH)_4$ (serpentine) + $Mg(OH)_2$ (brucite)

Reaction 1 produces weathering-resistant silica-bearing serpentinite that forms pinnacles, common in the New Idria serpentinite, whereas reaction 2 produces predominantly highly weathered and rounded boulders and fine soil-like material (Malcolm Ross, written commun., 1995). Serpentinization releases calcium that becomes available for secondary minerals. Iron in the primary silicate minerals, when not incorporated into serpentinite, may form magnetite. With increasing metamorphic temperature, lizardite of the original serpentinite is replaced by antigorite, which at higher temperatures is converted, by dehydration, to talc, olivine, and water (Coleman and Jove, 1993). Common alteration products in metamorphosed serpentinites of Quebec and Vermont are talc in steatite and schistose masses, talc-carbonate rocks, and quartz-carbonate bodies (Chidester and others, 1978).

Late stage serpentinization of carbonate-hosted deposits follows dedolomitization and development of calcium-magnesium silicate minerals during contact metamorphism. Subsequent serpentine alteration proceeds as outlined for serpentinite-hosted deposits.

Nature of ore

Serpentine-hosted deposits: These deposits commonly consist of stockworks (networks of veins) of cross-fiber veins that are aggregates of two or more thin parallel layers of fibers oriented about normal to vein walls. Most deposits contain some slip-fiber veins composed of fibers in the plane of the fracture. In a few deposits, this is the principal fiber type. In the New Idria serpentinite, ore is in sheared and pulverized serpentinite and consists of flake-like agglomerates and sheet-like masses of finely matted chrysotile (Mumpton and Thompson, 1975). Much of this ore may be secondary (see section below entitled "Secondary mineralogy"). Asbestos contents vary widely in the serpentine-hosted deposit type. Typical deposits contain 5 to 15 volume percent asbestos fiber; in the New Idria serpentinite, ore contains in excess of 50 volume percent chrysotile (Mumpton and Thompson, 1975).
Carbonate-hosted deposits: Ore zones consist of sets of cross-fiber veins subparallel to bedding in the serpentinized host. Commonly, veins are composed of multiple parallel layers in which the fibers are oriented about at right angles to vein walls and aggregate 1 to 20 cm in width; multiple layers indicate incremental development. Veins pinch and swell, anastomose, and vary in dip. Productive zones may be 10 cm to a few decimeters thick and constitute as much as 40 volume percent of a serpentinite body (Shride, 1969; Otton and others, 1981). However, asbestos veins commonly are contained in much greater thicknesses of massive, barren serpentinite. Chrysotile veins locally are in fine-grained calc-silicate rocks developed from cherty dolomite.

Deposit trace element geochemistry

Manganese, chromium (trivalent), copper, nickel, and platinum are rare elements found in chrysotile (Ross, 1981; Wicks and O'Hanley, 1988). Hexavalent chromium, known to be toxic, is not present in these asbestos deposits. Trace element composition has not been considered an environmentally significant aspect of chrysotile deposits.

Ore and gangue mineralogy and zonation

In serpentine- and carbonate-hosted deposits, the only ore mineral is chrysotile. Serpentinite-hosted deposits contain magnetite in the serpentine gangue and locally talc, as well as quartz-calcite veins (Riordon, 1957). In Canadian deposits, magnetite commonly is concentrated in wall rock adjacent to asbestos veins, along vein walls, in partings between layers of multiple veins, and can be disseminated parallel to chrysotile fibers (Riordon, 1957). Chrysotile, brucite, and magnetite are common in serpentine of the New Idria body (Coleman, in press). Carbonate-hosted deposits may include sparse calcite veins, otherwise the gangue is serpentine.

Mineral characteristics

Chrysotile is one of six mineral species called asbestos because of their fibrous habit (Skinner and others, 1988). Of these, chrysotile is the only fibrous serpentine mineral. The other five asbestos minerals belong to the amphibole group; these are grunerite asbestos (commonly referred to as amosite), riebeckite asbestos (commonly referred to as crocidolite), anthophyllite asbestos, tremolite asbestos, and actinolite asbestos. Chrysotile ($Mg_3Si_2O_5(OH)_4$) consists of layers of linked $SiO_4$ tetrahedra and misfit layers of linked $MgO_2(OH)_4$ tetrahedra that together roll into sheets, making hollow tubes having diameters of about 25 nm (Ross, 1981). Most chrysotile contains less than 2 weight percent iron as FeO, though as much as 8 percent has been reported (Wicks and O'Hanley, 1988). Small amounts of aluminum, manganese, magnesium, calcium, potassium, and sodium also may be present (Ross and others, 1984). Vein fibers range in length from less than 5 µm to 10 cm or more. They can vary in flexibility, hardness, tensile strength, and other physical properties and chemical properties of importance in determining their commercial use (Shride, 1973).

Secondary mineralogy

Although serpentine minerals are considered to be stable in the upper crust (Coleman and Jove, 1993), studies in California show that they are unstable in the range of pH and $Mg^{2+}$ and $Si(OH)_4$ concentrations encountered in most soil (Mumpton and Thompson, 1966; Wildman and others, 1971). Iron-rich montmorillonite is a common product of serpentine in this soil type (Wildman and others, 1971). Brucite ($Mg(OH)_2$), one of the hydrothermal minerals that develops during serpentinization and makes up 7 to 8 volume percent of the New Idria serpentinite, is destroyed in the weathering zone, producing coalingite (an iron- and magnesium-bearing carbonate), compositionally similar pyroaurtite (Mumpton and Thompson, 1966), hydromagnesite, and secondary chrysotile (Mumpton and Thompson, 1975). Some chrysotile is developed during tectonic milling and may be the most important process in the formation of chrysotile at the New Idria deposit (R.G. Coleman, written commun., 1995). Deeply weathered serpentinite can produce the nickel-bearing minerals nepouite and pecoraite (Mumpton and Thompson, 1975), but most serpentine contains only very small amounts of nickel.

Tremolite, an amphibole, can be fibrous and is associated with chrysotile deposits, particularly at Thetford Mines, Quebec; Troodos, Cyprus; and Balangero, Italy (R.G. Coleman, written commun., 1995). The EPA classifies tremolite as asbestos if the particles have an aspect ratio of 3:1. The amount and distribution of tremolite asbestos in chrysotile deposits is poorly known. It is extremely scarce in the New Idria, Calif., deposits (Coleman, in press), and it makes up less than one volume percent of the dust in Quebec mines and mills (Mossman and others, 1990).

Topography, physiography

Serpentine-hosted deposits: These deposits are easily eroded. Serpentinite bodies commonly are well exposed and therefore are readily eroded. The asbestos deposit at Belvidere Mountain, Vt., is on a hillside in an area having relief of 200 m in a radius of 1.6 km (Chidester and others, 1951). The New Idria serpentinite body in San Benito County, Calif., occupies a high ridge, as illustrated in Mumpton and Thompson (1975), that includes rock exposures barren of vegetation. Evidence of contact dislocation shows that this serpentinite body is rising tectonically, exposing this soft material to long term erosion (Coleman, in press).

Carbonate-hosted deposits: Chrysotile-bearing serpentinite zones in Arizona are soft and weakly resistant to weathering. However, these zones commonly are exposed in cliffs and steep slopes, protected by the more resistant, overlying metalimestone beds, which reduces exposure of serpentinite and chrysotile to erosion.

Hydrology

Streams draining asbestos-bearing serpentinite can pick up chrysotile fibers. Most streams draining the Belvidere Mountain deposit, Vt. lead to the Gihon and Lamoille Rivers and Lake Champlain, but some reach Quebec via the Missisquoi River. Stream drainages from the northeast and southeast parts of the New Idria serpentinite, Calif., lead

to the San Joaquin Valley. This area drains to the west via the San Benito River to the Pacific Ocean. Debris slides along the flanks of the serpentinite body and in the main drainage channels contain huge amounts of asbestos-bearing material available for removal and dispersion by streams (Cowan, 1979). During flood stage, streams flowing into the San Joaquin Valley from the New Idria mass have introduced sediment into the California aqueduct (Coleman, in press). Asbestos fibers also have been found in the water supply for San Francisco (Kanarek and others, 1980). Drainage from carbonate-type chrysotile deposits in Arizona reaches the watershed of the Salt River, which flows to Phoenix via several reservoirs.

Mining and milling methods

Serpentine-hosted deposits: Exploitation of these deposits is mostly by open-pit mining. Block caving and other underground methods have been used. Milling generally is a dry process and follows practices used, slightly modified, since the early days of this century (Mann, 1983). Ore is crushed, screened, and dried. Drying is accomplished using hot air, which is filtered after the fibers are dried. Processing and packaging methods have improved significantly in recent years, which makes it possible to handle asbestos in a nearly dust-free environment. Most producers grade and classify milled asbestos fibers as a function of fiber length, using a standard developed in Canada (Energy, Mines, and Resources Canada, 1976; Mann, 1983). A wet process for milling chrysotile was installed in 1990 at the Baie Verte Mines in Newfoundland (Stewart and others, 1990). In this process, mill feed is wet screened, classified, then dried in a propane-fired dryer. Impurities are collected in a thickener and pumped to tailings dumps. A wet process was used in 1991 at the KCAC Inc. mine in San Benito County, Calif, (Virta, 1991).

Carbonate-hosted deposits: Chrysotile mining in Arizona has been by small-scale underground methods using adits, room and pillar methods, and stoping with backfill. Milling of chrysotile ore in Arizona was stopped by court order in 1974. The milling process consisted of crushing the ore, beating it to free the fibers, then removing the fibers by screening and air separation (Bowles, 1955). Waste rock and unwanted fibers were consigned to tailings dumps.

ENVIRONMENTAL SIGNATURES

Drainage signatures

Stream channels that drain chrysotile-bearing serpentinite contain asbestos as a natural erosion product. Where serpentinite masses crop out in mountainous terrane, as in San Benito County, Calif., chrysotile-bearing debris in landslides, debris flows, and bedrock exposures provide extensive sources of asbestos materials to local streams. Some of these fibers have reached the California aqueduct on the west side of the San Joaquin Valley.

Asbestos mobility from solid mine wastes

Mine dumps and mill tailings at chrysotile deposits are sources of asbestos fibers for surface water and are more easily eroded than outcrops.

Soil, sediment signatures prior to mining

Soil developed on asbestos-bearing bedrock may contain asbestos fibers. The soil over the New Idria body is composed of chrysotile and brucite with some lizardite and (or) antigorite, plus minor amounts of other minerals.

Potential environmental concerns associated with mineral processing

The principal concerns include dust and tailings from asbestos milling operations. Asbestos fibers in tailings are available for airborne and fluvial transport.

Smelter signatures

Not applicable to asbestos.

Climate effects on environmental signatures

Where serpentinization is ongoing, meteoric water becomes saturated with calcium hydroxide and can precipitate brucite, portlandite, and carbonate minerals. In arid climates, weathering can lead to the precipitation of brucite, carbonate minerals, chrysotile, and talc (Coleman and Jove, 1993). In humid conditions, serpentinite can be dissolved leaving a residue rich in iron and smectite.

Chrysotile in some environments is known to survive for thousands to millions of years after being subjected to erosion and transport to distant sites. For example, accumulations of chrysotile fibers eroded from the New Idria,

Calif., serpentinite have been identified in downstream sedimentary accumulations such as terrace deposits and alluvial fans (Coleman, in press). Chrysotile also is preserved in the Big Blue Formation of Miocene age, which contains abundant debris eroded from the New Idria serpentinite (Carlson, 1981).

Geoenvironmental geophysics

See exploration geophysics.

## CONTROVERSY REGARDING HEALTH RISKS TO HUMANS FROM CHRYSOTILE ASBESTOS

The risk of asbestos to human health has been known at least since 1906 when workers in an asbestos weaving mill in France died (D'Agostino and Wilson, 1993). In the 1920s, a death from asbestosis (fibrosis of the lung) was reported and the disease was named (Sawyer, 1987). However, not until the 1960s were the biologic effects of asbestos fibers documented in great detail and a relationship clearly established between exposure to asbestos and lung disease, including cancer. As a result of concern developed from increasing knowledge of health hazards associated with asbestos minerals, the Occupational Safety and Health Administration (OSHA) in 1971 issued regulations restricting airborne asbestos in the workplace, and in 1987 OSHA established the current workplace standard of 0.2 fibers per cubic centimeter of air. However, the regulations do not discriminate between fibers of the different asbestos minerals. Since the establishment of these regulations, numerous studies have demonstrated that various asbestos minerals have significantly different associated health effects. For example, the few cases of mesothelioma (cancer of the pleura or peritoneum) in Canadian chrysotile miners and mill workers appear to be not from chrysotile but perhaps from small amounts of tremolite asbestos, and a study of British workers engaged in the manufacture of friction materials using only chrysotile showed no excess of deaths from lung diseases (Mossman and others, 1990). Health effects related to occupational exposure to chrysotile are the least of any asbestos mineral; people exposed to chrysotile alone, at abundances more than 10 times higher than recommended by the EPA, experience no excess lung cancer (Coleman, in press).

## RISK ASSESSMENT

Hazards resulting from inhalation of asbestos fibers have been documented by the EPA and have been the topic of considerable scientific inquiry (Ross, 1981, 1984; Skinner and others, 1988; Mossman and others, 1990; D'Agostino and Wilson, 1993; Ross and Skinner, 1994; McDonald and McDonald, 1995). Although the relationship between asbestos and lung diseases is well documented, debate continues regarding the risk from low level exposure to asbestos fiber. In a report prepared for the California Environmental Protection Agency, risk related to asbestos was not ranked because data on low level exposure were considered inadequate (California Comparative Risk Project, 1994). Studies show that important factors to be considered in evaluating risk associated with asbestos inhalation include type of asbestos mineral, length and diameter of the asbestos fibrils, amount of asbestos inhaled, and the duration of the exposure. Yet, despite conclusions that risks from chrysotile asbestos are almost certainly lower than for other asbestos minerals (D'Agostino and Wilson, 1993), uncertainty in the degree of risk from exposure to chrysotile remains. The uncertainty results in part from disagreement concerning whether an exposure threshold exists and, if so, at what fiber concentration below which inhalation is safe (D'Agostino and Wilson, 1993).

The EPA has concluded that inhalation of any amount of asbestos is potentially hazardous, that a single asbestos fiber can be lethal (Abelson, 1990). This conclusion results from belief that a linear relationship exists between asbestos dose and health risk such that risk exists even at very low levels of exposure (D'Agostino and Wilson, 1993). According to this theory, any exposure to asbestos poses a risk. In a nonlinear relation, risk from exposure decreases rapidly at low levels and a threshold value can be reached below which the risk is zero. Recent studies suggest that low-level exposure to chrysotile asbestos in the environment has generated unwarranted concern based on speculation (D'Agostino and Wilson, 1993) and that the single-fiber view is unproved (Abelson, 1990). Other studies suggest that a "threshold" value, below which exposure to chrysotile asbestos causes no measurable health effects, can be identified (Ross, 1987).

Estimates of risk to human health from numerous activities, including everyday risks, have been quantified, and a few attempts have been made to quantify risk of exposure to chrysotile asbestos under different environmental conditions (D'Agostino and Wilson, 1993; Coleman, in press). For example, data show that risks from inhaling asbestos during recreational activities at the chrysotile-bearing New Idria, Calif., serpentinite or from exposure to asbestos in schools are low. Coleman (1995) concluded that "the apparent risk in making one trip by automobile to New Idria is 300 times greater than inhaling [chrysotile] fibers during a lifetime of recreation in this area." Risks from occupying schools containing chrysotile fibers are even lower and have been categorized as harmlessly small

(Abelson, 1990; Wilson and others, 1994).

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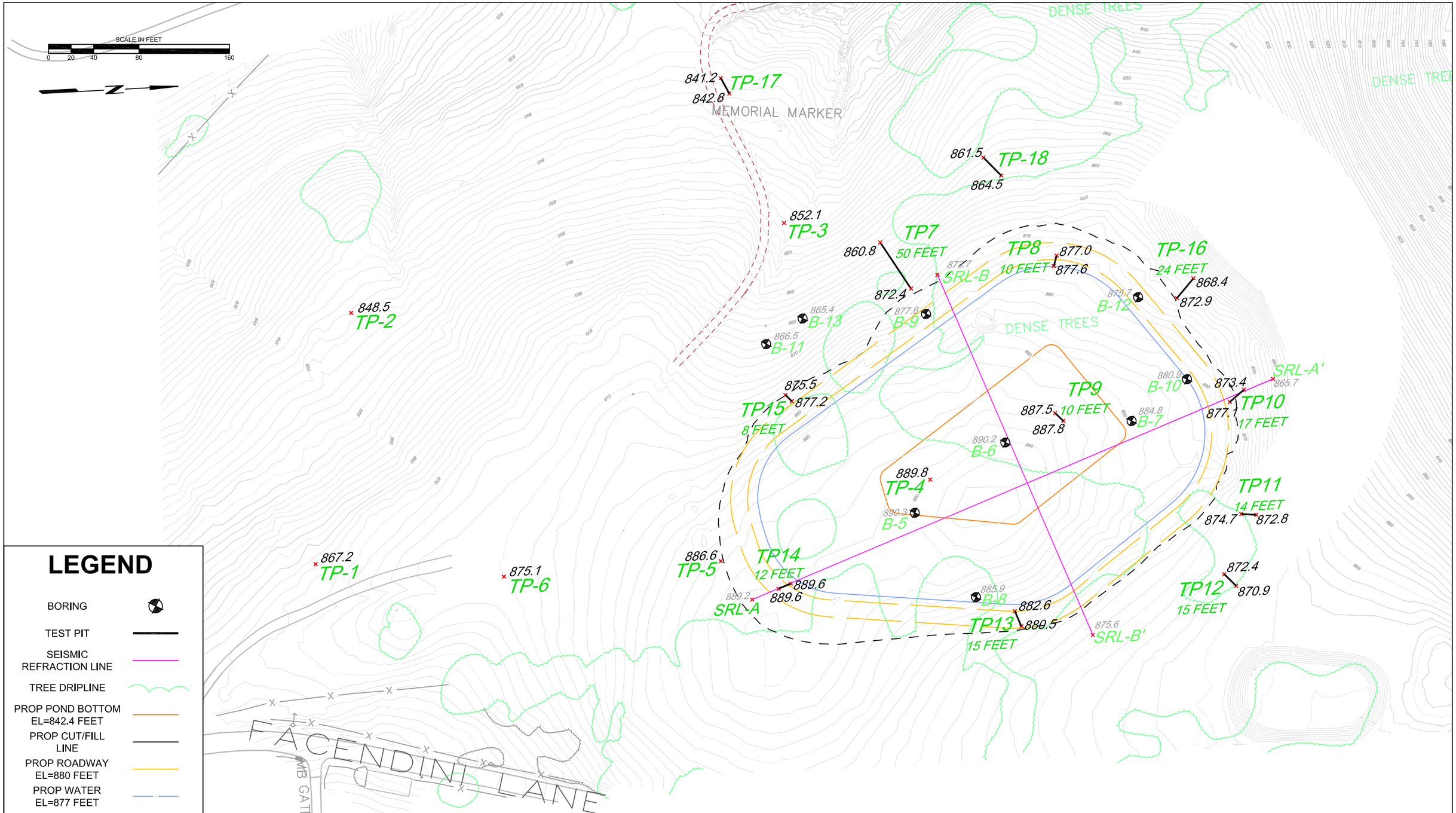
# APPENDIX H

# SITE 3C & 3C-2

# LOGS OF TEST PITS

# TP-1 THROUGH TP-18




## LEGEND

- BORING
- TEST PIT
- SEISMIC REFRACTION LINE
- TREE DRIPLINE
- PROP POND BOTTOM EL=842.4 FEET
- PROP CUT/FILL LINE
- PROP ROADWAY EL=880 FEET
- PROP WATER EL=877 FEET

| | |
|---|---|
| COMPILED BY: | N. SCHEEMAKER |
| REVISED BY: | N. SCHEEMAKER |
| CHECKED BY: | A. JOHNS |
| DATE: 01/05/2013 | APPROVED BY: Y. PRASHAR |

# SITE 3C-2 TEST PIT LOCATION PLAN

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / GEO 2

SCALE: 1 inch:80 feet

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

PLATE NO.
TEST PIT PLAN

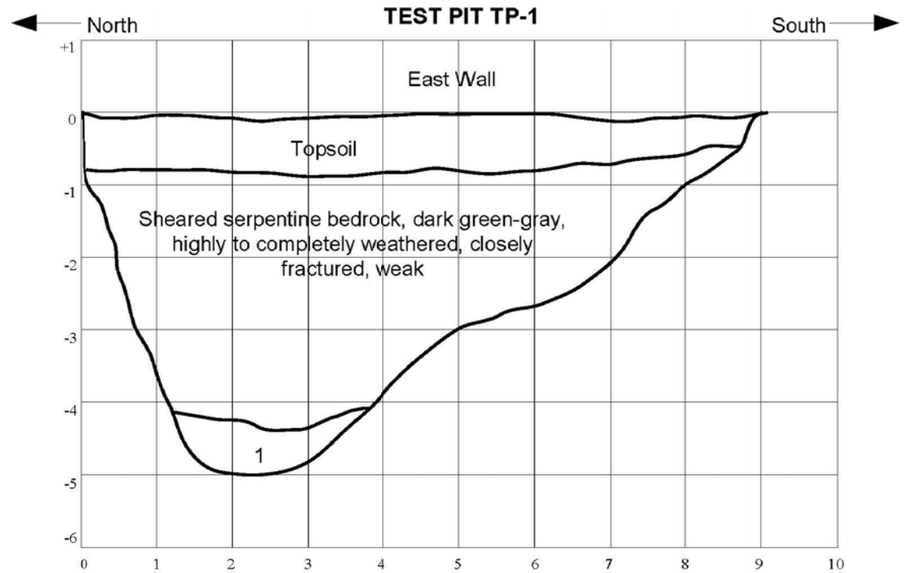
## GREAT VALLEY COMPLEX

**KJgv** **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.
*Primarily present in site 3C-2 as sandstone conglomerate, shale, meta shale & graywacke sequences*

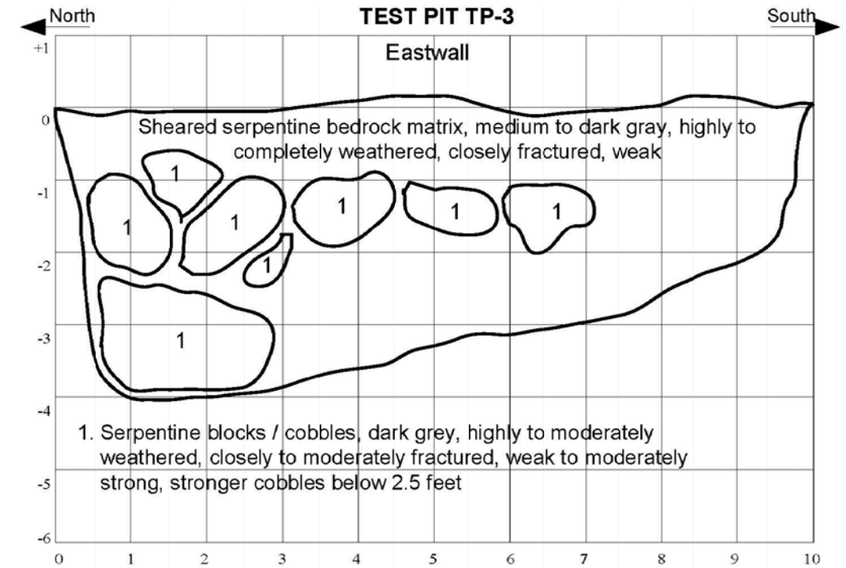
**Jsv** **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).
*Primarily present in site 3C-2 as isolated surface and near surface exotic blocks of intermediate tuff & meta tuff. Gray-green, very strong*

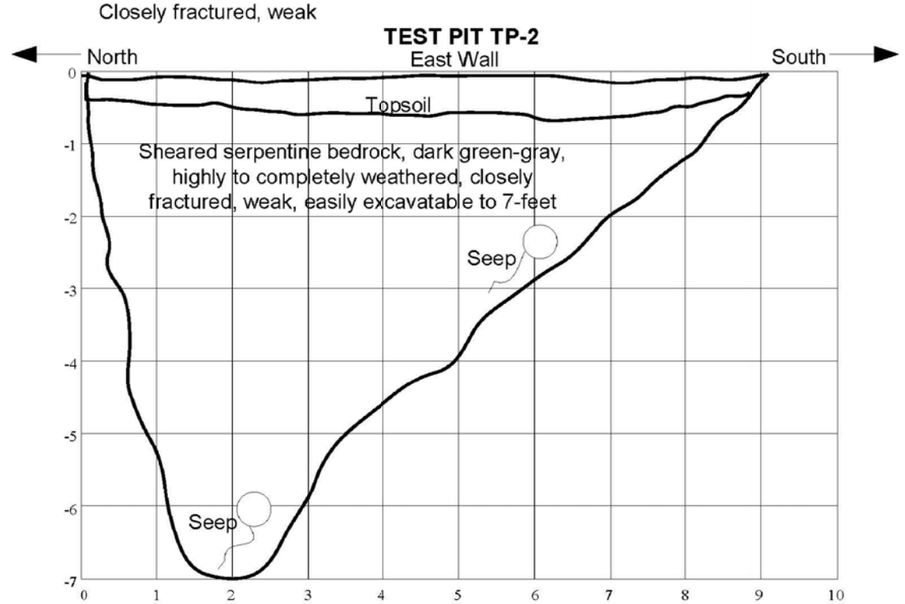
## COAST RANGE OPHIOLITE

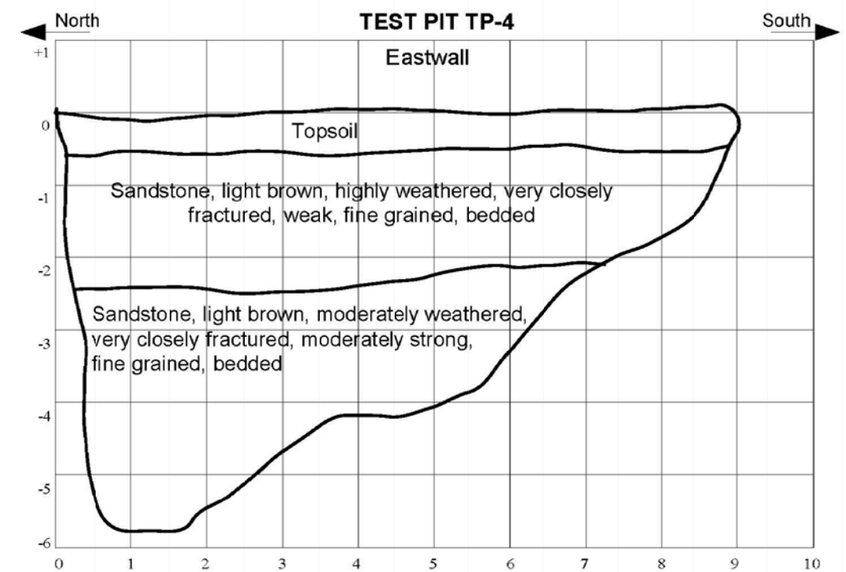
**spm** **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.
*Primarily present in site 3C-2 as highly variable sheared serpentinite*




1. Sheared serpentine bedrock, dark green-gray, high to moderately weathered, Closely fractured, weak




| | | | |
|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | TEST PITS 1 - 4 | M2 CONSULTANTS inc | PLATE NO. |
| REVISED BY: N. SCHEEMAKER | | | TP 1 - 4 |
| CHECKED BY: A. JOHNS | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/20/2013 / APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 — SCALE: 1 inch:2 feet | 638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | |

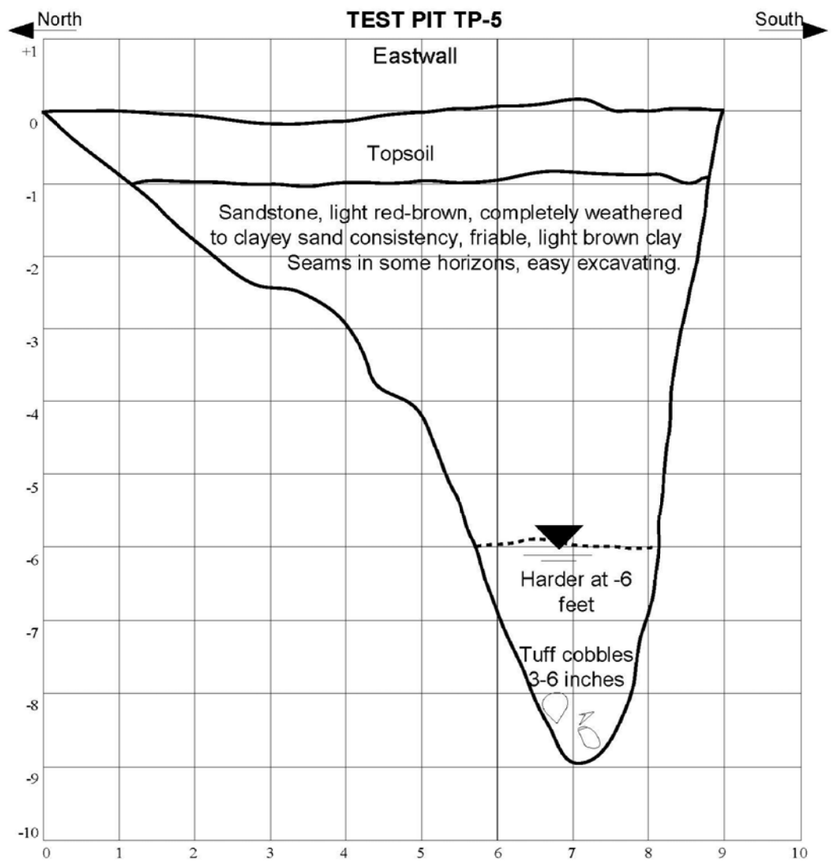
## GREAT VALLEY COMPLEX

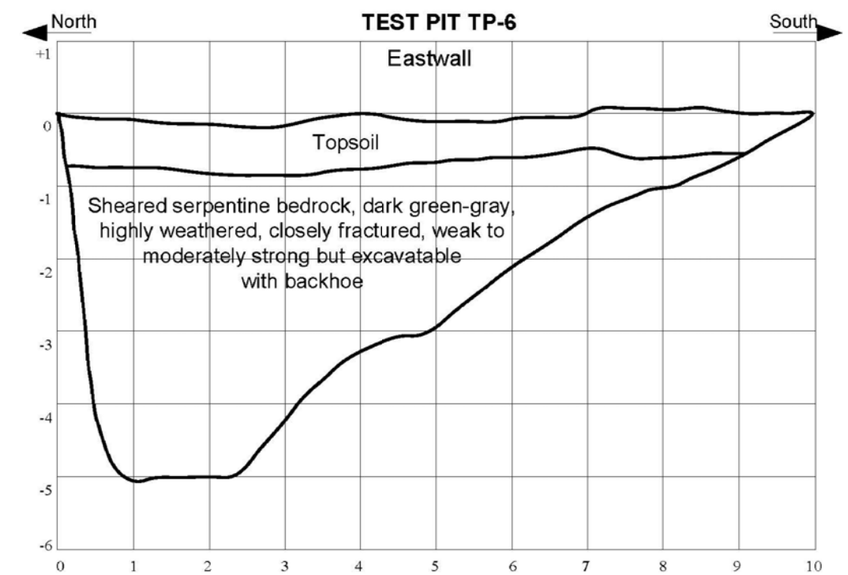
KJgv **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.
*Primarily present in site 3C-2 as sandstone conglomerate, shale, meta shale & graywacke sequences*

Jsv **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).
*Primarily present in site 3C-2 as isolated surface and near surface exotic blocks of intermediate tuff & meta tuff. Gray-green, very strong*

## COAST RANGE OPHIOLITE

spm **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.
*Primarily present in site 3C-2 as highly variable sheared serpentinite*




| | | | |
|---|---|---|---|
| COMPILED BY: N. SCHEEMAKER | TEST PITS 5 & 6 | M2 CONSULTANTS inc<br>638 19th STREET, UNIT 18<br>SAN FRANCISCO, CALIFORNIA, 94107 | PLATE NO.<br>TP 5&6 |
| REVISED BY: N. SCHEEMAKER | | | |
| CHECKED BY: A. JOHNS | OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT<br>OCCIDENTAL, CALIFORNIA | | |
| DATE: 01/20/2013 / APPROVED BY: Y. PRASHAR | PROJECT NO: 01202 / GEO 2 — SCALE: 1 inch:2 feet | | |

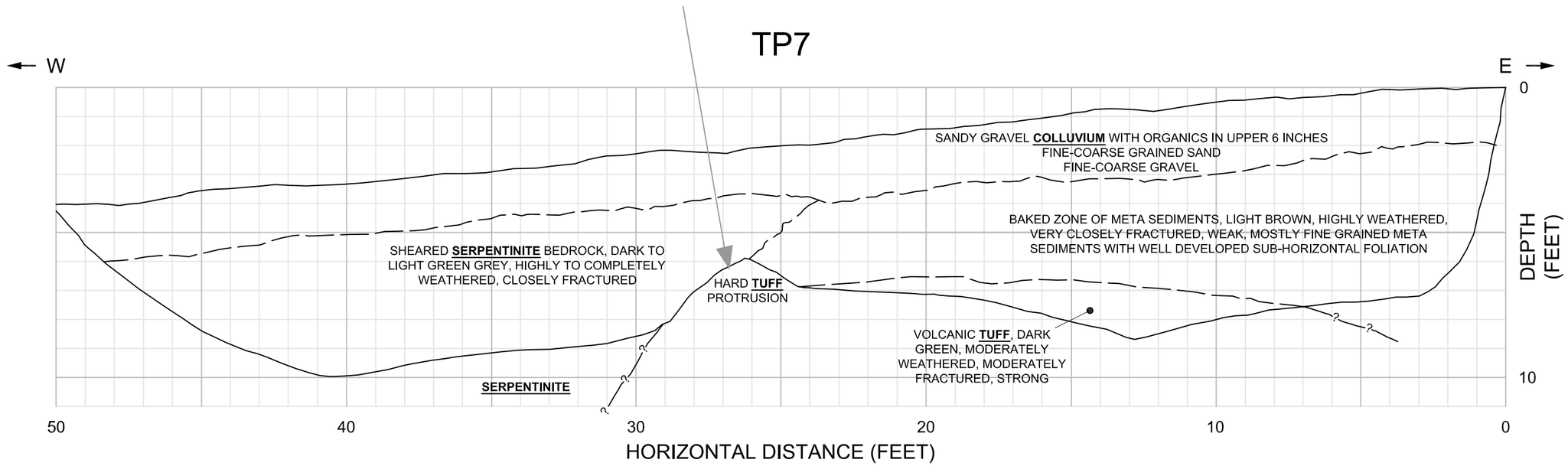
## GREAT VALLEY COMPLEX

**KJgv** **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.
*Primarily present in site 3C-2 as sandstone conglomerate, shale, meta shale & graywacke sequences*

**Jsv** **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).
*Primarily present in site 3C-2 as isolated surface and near surface exotic blocks of intermediate tuff & meta tuff. Gray-green, very strong*

## COAST RANGE OPHIOLITE

**spm** **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.
*Primarily present in site 3C-2 as highly variable sheared serpentinite*




| TEST PIT ID | TP7 | ORIENTATION | N30E |
|---|---|---|---|
| DATE EXCAVATED | 10/23/12 | WIDTH | 2 FEET |
| DATE BACKFILLED | 10/23/12 | MAX DEPTH | 10 FEET |
| LOGGED FACE | NW WALL | LENGTH | 50 FEET |

COMMENTS
- CONTACT BETWEEN SERPENTINITE, TUFF, AND SANDSTONE
- NO GROUNDWATER ENCOUNTERED
- WHEELROLLED TO EXISTING GROUND SURFACE

COMPILED BY: N. SCHEEMAKER
REVISED BY: N. SCHEEMAKER
CHECKED BY: A. JOHNS
DATE: 01/20/2013
APPROVED BY: Y. PRASHAR

# TEST PIT 7

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / GEO 2
SCALE: 1 inch:4 feet

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

PLATE NO.
TP 7

## GREAT VALLEY COMPLEX

**KJgv** — **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.
*Primarily present in site 3C-2 as sandstone conglomerate, shale, meta shale & graywacke sequences*

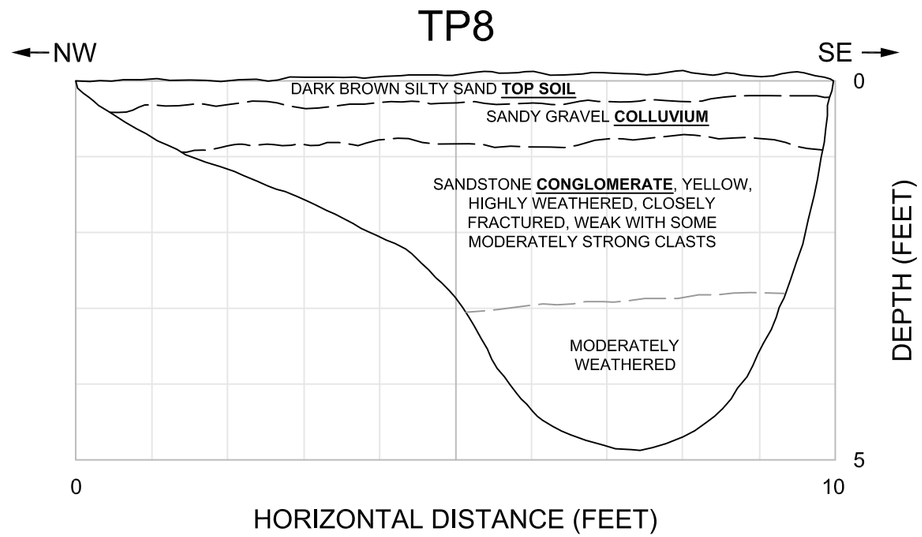
**Jsv** — **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).
*Primarily present in site 3C-2 as isolated surface and near surface exotic blocks of intermediate tuff & meta tuff. Gray-green, very strong*

## COAST RANGE OPHIOLITE

**spm** — **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.
*Primarily present in site 3C-2 as highly variable sheared serpentinite*

| TEST PIT ID | TP8 | ORIENTATION | S70W |
|---|---|---|---|
| DATE EXCAVATED | 10/23/12 | WIDTH | 2 FEET |
| DATE BACKFILLED | 10/23/12 | MAX DEPTH | 4.8 FEET |
| LOGGED FACE | N WALL | LENGTH | 10 FEET |

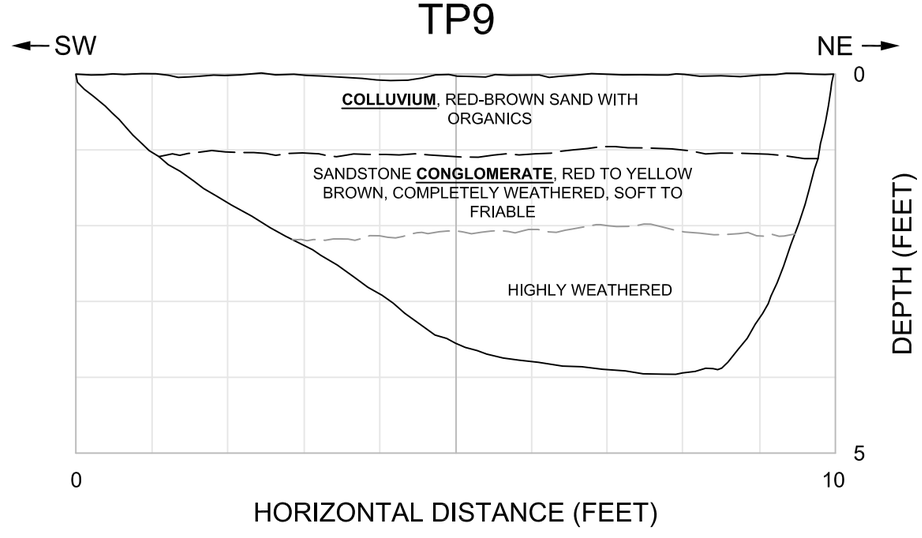
COMMENTS
- SANDSTONE CONGLOMERATE BEDROCK, (KJgv)
- NO GROUNDWATER ENCOUNTERED
- WHEELROLLED TO EXISTING GROUND SURFACE



TP-8A

| TEST PIT ID | TP9 | ORIENTATION | S20W |
|---|---|---|---|
| DATE EXCAVATED | 10/23/12 | WIDTH | 2 FEET |
| DATE BACKFILLED | 10/23/12 | MAX DEPTH | 4 FEET |
| LOGGED FACE | W WALL | LENGTH | 10 FEET |

COMMENTS
- SANDSTONE CONGLOMERATE BEDROCK, (KJgv)
- NO GROUNDWATER ENCOUNTERED
- WHEELROLLED TO EXISTING GROUND SURFACE



TP-9A

| | |
|---|---|
| COMPILED BY: | N. SCHEEMAKER |
| REVISED BY: | N. SCHEEMAKER |
| CHECKED BY: | A. JOHNS |
| DATE: 01/20/2013 | APPROVED BY: Y. PRASHAR |

# TEST PITS 8 & 9

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / GEO 2

SCALE: 1 inch:2 feet

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

PLATE NO.
TP 8&9

## GREAT VALLEY COMPLEX

**KJgv** — **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.
*Primarily present in site 3C-2 as sandstone conglomerate, shale, meta shale & graywacke sequences*

**Jsv** — **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).
*Primarily present in site 3C-2 as isolated surface and near surface exotic blocks of intermediate tuff & meta tuff. Gray-green, very strong*

## COAST RANGE OPHIOLITE

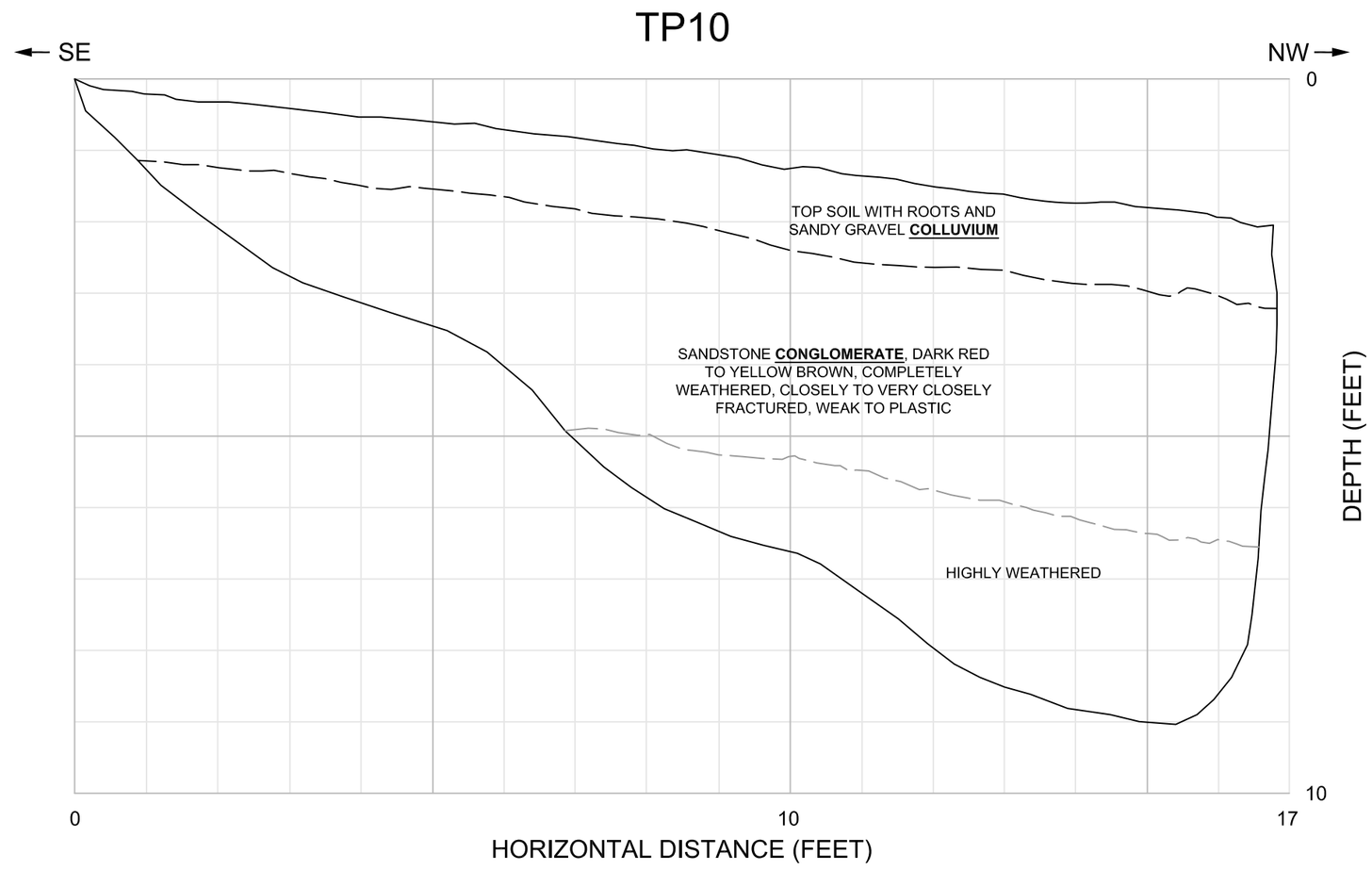
**spm** — **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.
*Primarily present in site 3C-2 as highly variable sheared serpentinite*

NW

TP-10A

NW

TP-10B




| | | | |
|---|---|---|---|
| TEST PIT ID | TP10 | ORIENTATION | N55W |
| DATE EXCAVATED | 10/23/12 | WIDTH | 2 FEET |
| DATE BACKFILLED | 10/23/12 | MAX DEPTH | 9 FEET |
| LOGGED FACE | SW WALL | LENGTH | 17 FEET |

COMMENTS
- SANDSTONE CONGLOMERATE BEDROCK, (KJgv)
- NO GROUNDWATER ENCOUNTERED
- WHEELROLLED TO EXISTING GROUND SURFACE

| | |
|---|---|
| COMPILED BY: N. SCHEEMAKER | |
| REVISED BY: N. SCHEEMAKER | |
| CHECKED BY: A. JOHNS | |
| DATE: 01/20/2013 | APPROVED BY: Y. PRASHAR |

# TEST PIT 10

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / GEO 2

SCALE: 1 inch:2 feet

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

PLATE NO.
TP 10

## GREAT VALLEY COMPLEX

**KJgv** — **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.
*Primarily present in site 3C-2 as sandstone conglomerate, shale, meta shale & graywacke sequences*

**Jsv** — **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).
*Primarily present in site 3C-2 as isolated surface and near surface exotic blocks of intermediate tuff & meta tuff. Gray-green, very strong*

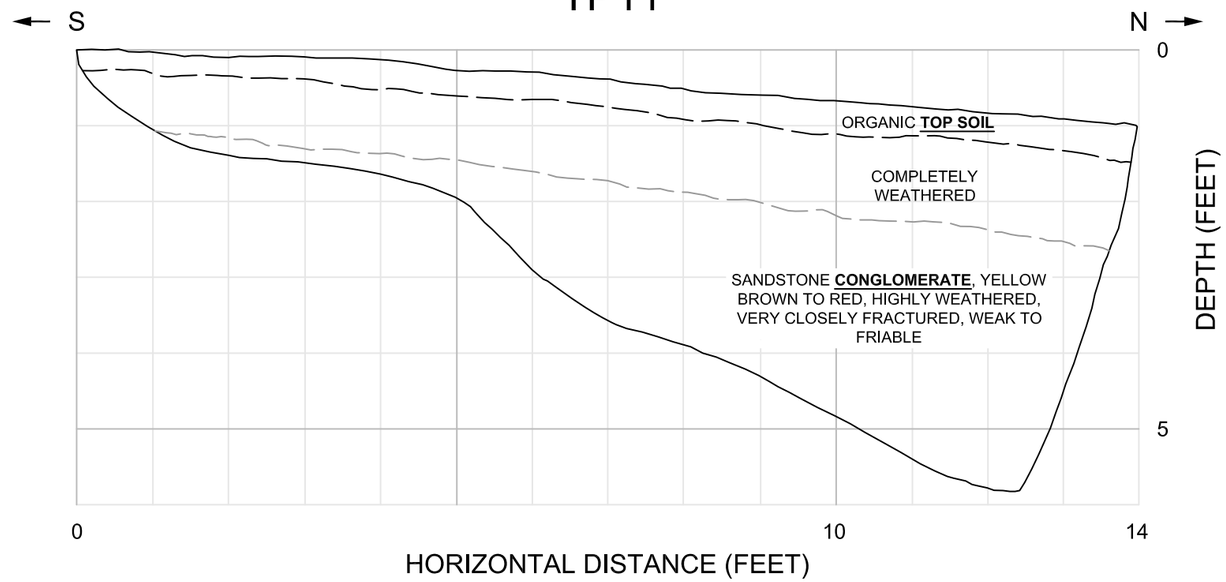
## COAST RANGE OPHIOLITE

**spm** — **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.
*Primarily present in site 3C-2 as highly variable sheared serpentinite*

| TEST PIT ID | TP11 | ORIENTATION | S20E |
|---|---|---|---|
| DATE EXCAVATED | 10/23/12 | WIDTH | 2 FEET |
| DATE BACKFILLED | 10/23/12 | MAX DEPTH | 5.8 FEET |
| LOGGED FACE | W WALL | LENGTH | 12 FEET |

COMMENTS
- SANDSTONE CONGLOMERATE BEDROCK, (KJgv)
- NO GROUNDWATER ENCOUNTERED
- WHEELROLLED TO EXISTING GROUND SURFACE

## TP11

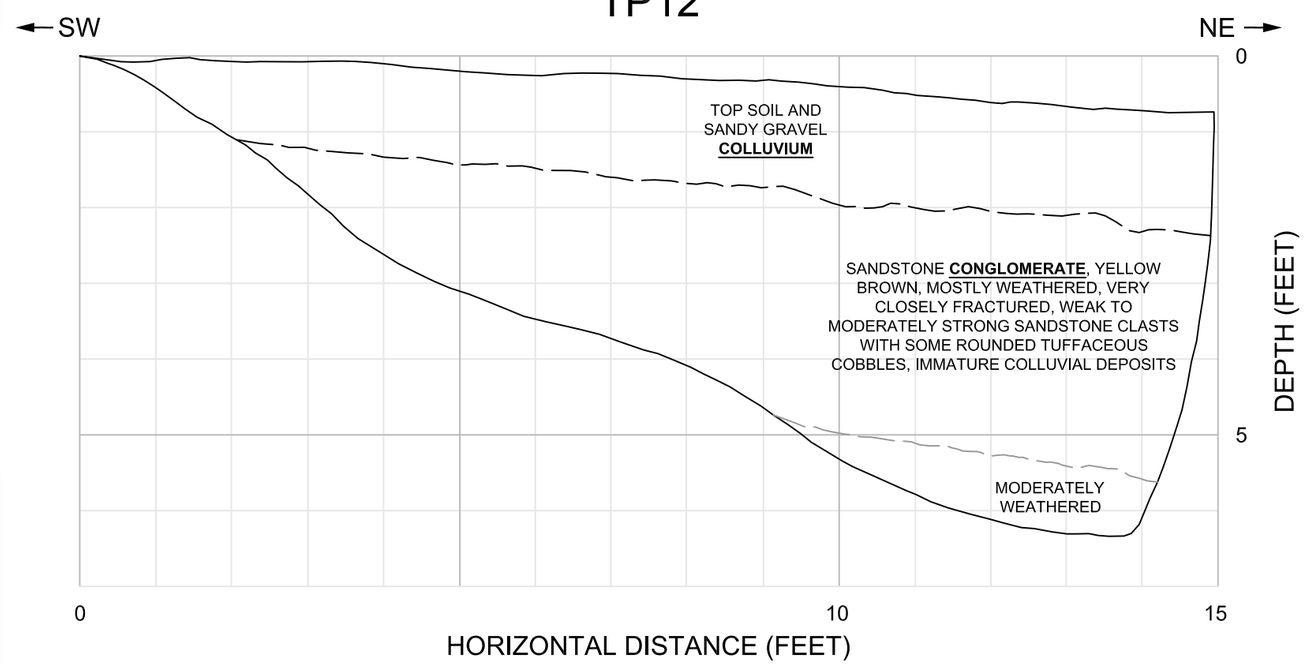


TP-11A

| TEST PIT ID | TP12 | ORIENTATION | S20W |
|---|---|---|---|
| DATE EXCAVATED | 10/23/12 | WIDTH | 2 FEET |
| DATE BACKFILLED | 10/23/12 | MAX DEPTH | 6.3 FEET |
| LOGGED FACE | NW WALL | LENGTH | 15 FEET |

COMMENTS
- SANDSTONE CONGLOMERATE BEDROCK, (KJgv)
- NO GROUNDWATER ENCOUNTERED
- WHEELROLLED TO EXISTING GROUND SURFACE

## TP12



TP-12A

| | |
|---|---|
| COMPILED BY: | N. SCHEEMAKER |
| REVISED BY: | N. SCHEEMAKER |
| CHECKED BY: | A. JOHNS |
| DATE: 01/20/2013 | APPROVED BY: Y. PRASHAR |

# TEST PITS 11 & 12

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / GEO 2

SCALE: 1 inch:2 feet

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

PLATE NO.
TP 11&12

## GREAT VALLEY COMPLEX

**KJgv** — **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.
*Primarily present in site 3C-2 as sandstone conglomerate, shale, meta shale & graywacke sequences*

**Jsv** — **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).
*Primarily present in site 3C-2 as isolated surface and near surface exotic blocks of intermediate tuff & meta tuff. Gray-green, very strong*

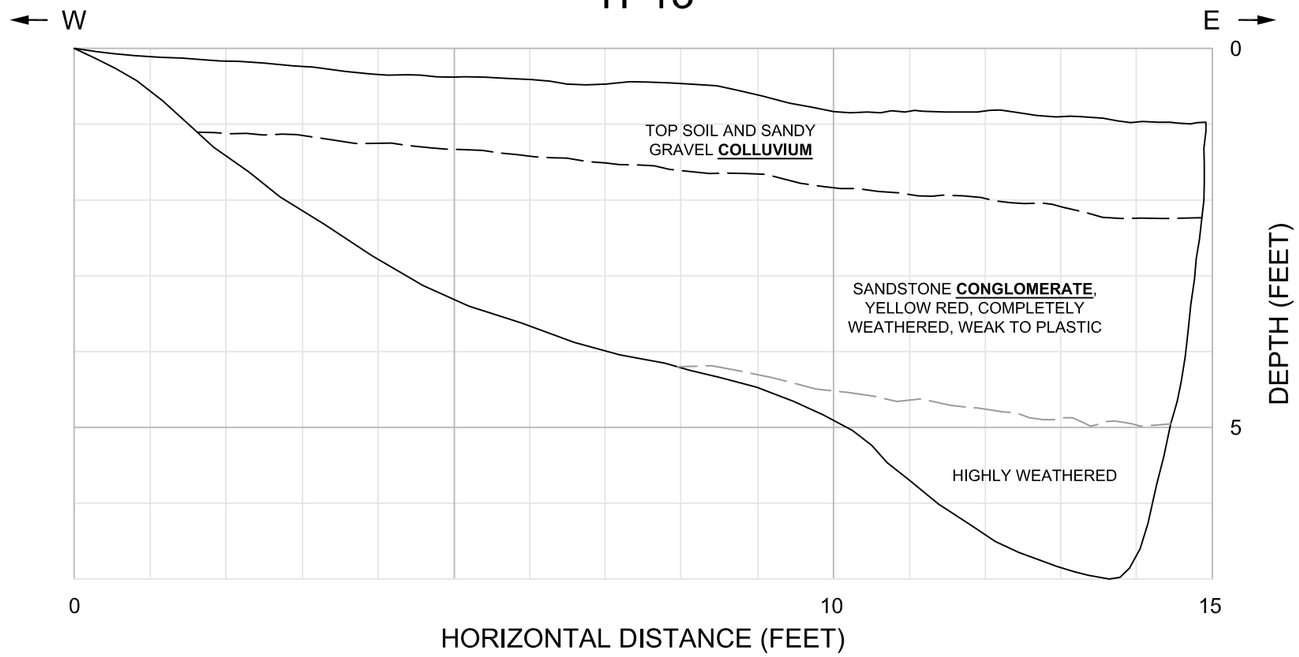
## COAST RANGE OPHIOLITE

**spm** — **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.
*Primarily present in site 3C-2 as highly variable sheared serpentinite*

| TEST PIT ID | TP13 | ORIENTATION | S45W |
|---|---|---|---|
| DATE EXCAVATED | 10/23/12 | WIDTH | 2 FEET |
| DATE BACKFILLED | 10/23/12 | MAX DEPTH | 7 FEET |
| LOGGED FACE | NW WALL | LENGTH | 15 FEET |

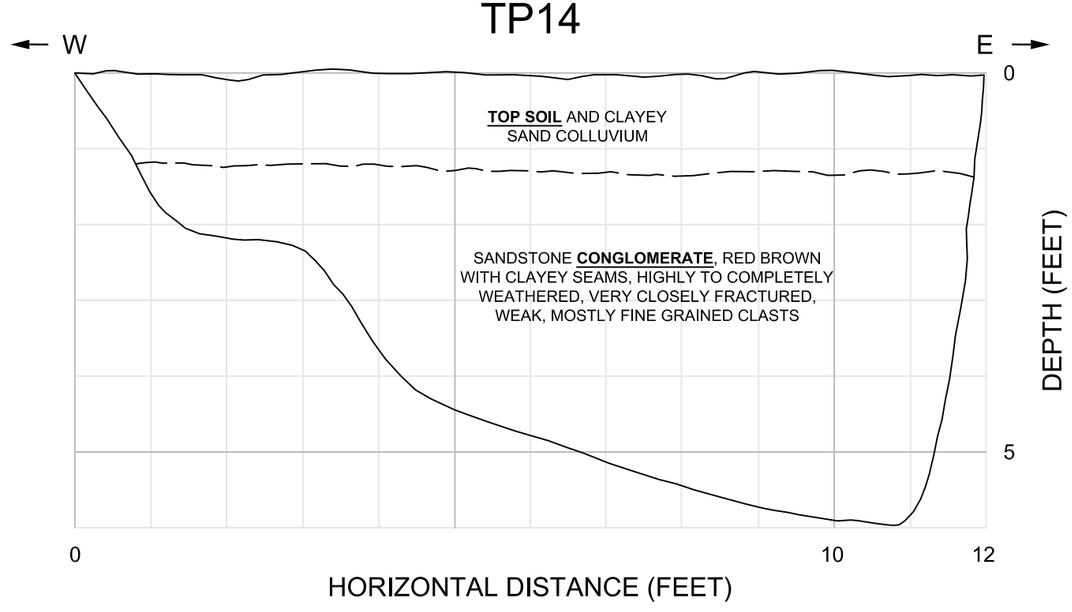
COMMENTS
- SANDSTONE CONGLOMERATE BEDROCK, (KJgv)
- NO GROUNDWATER ENCOUNTERED
- WHEELROLLED TO EXISTING GROUND SURFACE

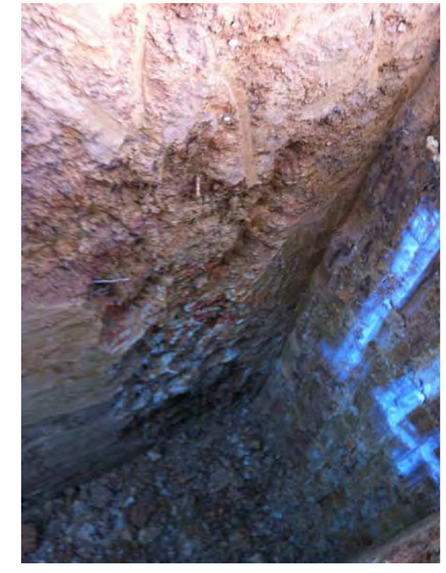
## TP13



TP-13A

## TP14



TP-14A

| TEST PIT ID | TP14 | ORIENTATION | N50W |
|---|---|---|---|
| DATE EXCAVATED | 10/23/12 | WIDTH | 2 FEET |
| DATE BACKFILLED | 10/23/12 | MAX DEPTH | 5.9 FEET |
| LOGGED FACE | NE WALL | LENGTH | 12 FEET |

COMMENTS
- SANDSTONE CONGLOMERATE BEDROCK, (KJgv)
- NO GROUNDWATER ENCOUNTERED
- WHEELROLLED TO EXISTING GROUND SURFACE

| | |
|---|---|
| COMPILED BY: | N. SCHEEMAKER |
| REVISED BY: | N. SCHEEMAKER |
| CHECKED BY: | A. JOHNS |
| DATE: 01/17/2013 | APPROVED BY: Y. PRASHAR |

# TEST PITS 13 & 14

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / GEO 2

SCALE: 1 inch:2 feet

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

PLATE NO.
TP 13&14

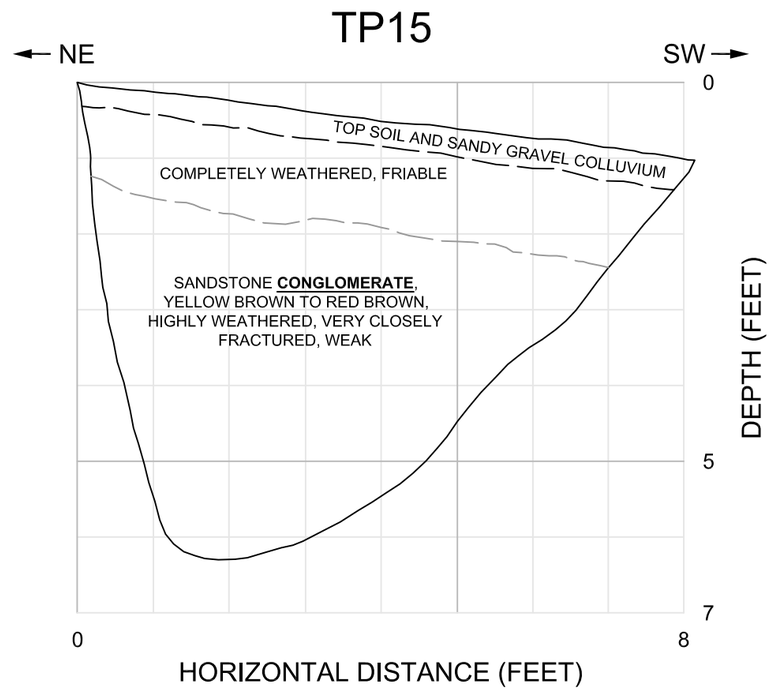
## GREAT VALLEY COMPLEX

**KJgv** **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.
*Primarily present in site 3C-2 as sandstone conglomerate, shale, meta shale & graywacke sequences*

**Jsv** **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).
*Primarily present in site 3C-2 as isolated surface and near surface exotic blocks of intermediate tuff & meta tuff. Gray-green, very strong*

## COAST RANGE OPHIOLITE

**spm** **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.
*Primarily present in site 3C-2 as highly variable sheared serpentinite*



TP-15A

TP-15B

| | | | |
|---|---|---|---|
| TEST PIT ID | TP15 | ORIENTATION | S20W |
| DATE EXCAVATED | 10/23/12 | WIDTH | 2 FEET |
| DATE BACKFILLED | 10/23/12 | MAX DEPTH | 6.3 FEET |
| LOGGED FACE | NW WALL | LENGTH | 8 FEET |

COMMENTS
- SANDSTONE CONGLOMERATE BEDROCK, (KJgv)
- NO GROUNDWATER ENCOUNTERED
- WHEELROLLED TO EXISTING GROUND SURFACE

COMPILED BY: N. SCHEEMAKER
REVISED BY: N. SCHEEMAKER
CHECKED BY: A. JOHNS
DATE: 01/20/2013
APPROVED BY: Y. PRASHAR

# TEST PIT 15

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / GEO 2
SCALE: 1 inch:2 feet

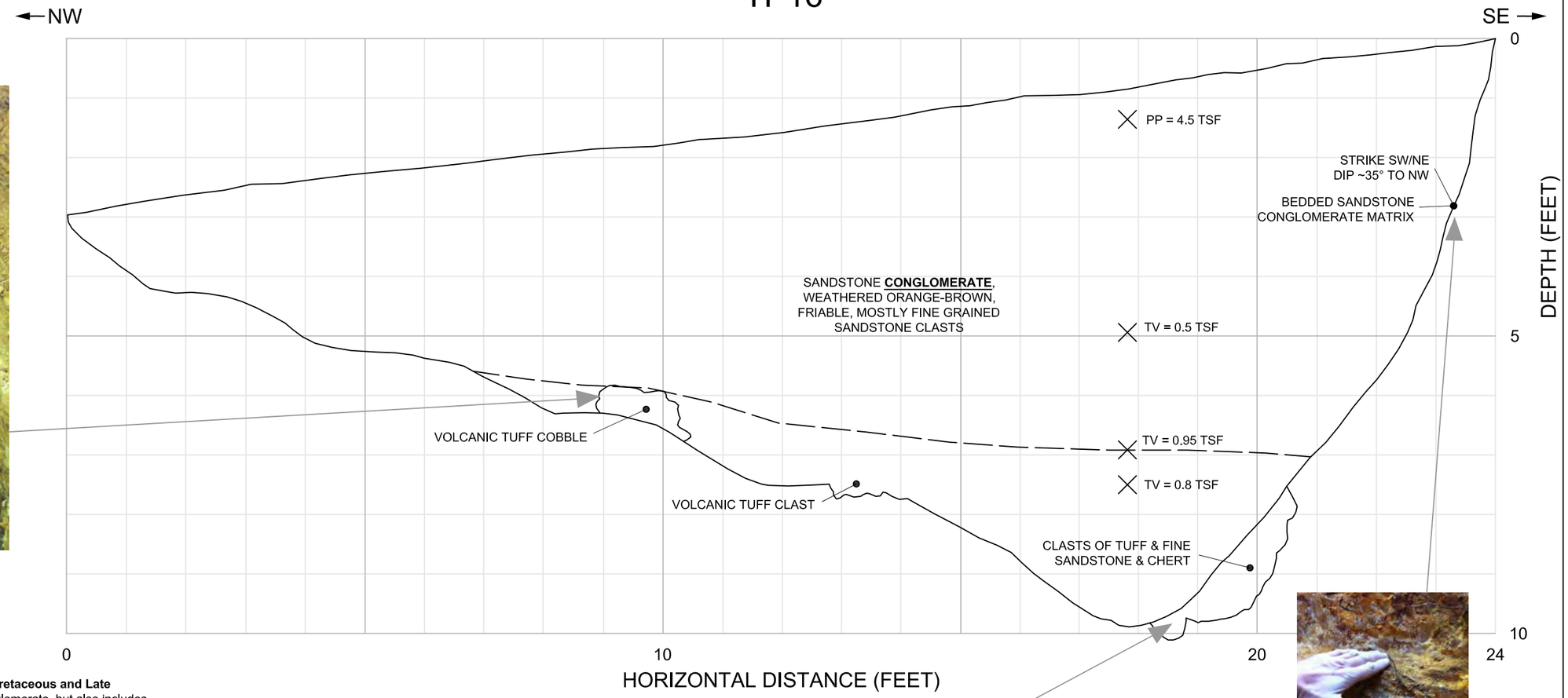
M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

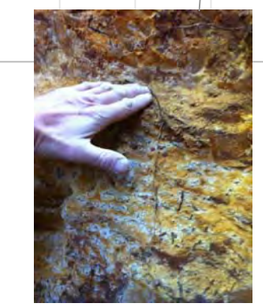
PLATE NO.
TP15

PP POCKET PENETROMETER READING
TV TORVANE READING

# TP16

TP-16A



TP-16D

## GREAT VALLEY COMPLEX

**KJgv** **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.
*Primarily present in site 3C-2 as sandstone conglomerate, shale, meta shale & graywacke sequences*

**Jsv** **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).
*Primarily present in site 3C-2 as isolated surface and near surface exotic blocks of intermediate tuff & meta tuff. Gray-green, very strong*

## COAST RANGE OPHIOLITE

**spm** **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.
*Primarily present in site 3C-2 as highly variable sheared serpentinite*

TP-16

TP-16B

TP-16C

| | | | |
|---|---|---|---|
| TEST PIT ID | TP16 | ORIENTATION | N50W |
| DATE EXCAVATED | 11/14/12 | WIDTH | 2 FEET |
| DATE BACKFILLED | 11/14/12 | MAX DEPTH | 10 FEET |
| LOGGED FACE | NE WALL | LENGTH | 24 FEET |

COMMENTS
- SANDSTONE CONGLOMERATE BEDROCK, (KJgv)
- NO GROUNDWATER ENCOUNTERED
- WHEELROLLED TO EXISTING GROUND SURFACE

| | |
|---|---|
| COMPILED BY: | N. SCHEEMAKER |
| REVISED BY: | N. SCHEEMAKER |
| CHECKED BY: | A. JOHNS |
| DATE: 01/20/2013 | APPROVED BY: Y. PRASHAR |

# TEST PIT 16

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / GEO 2

SCALE: 1 inch:2 feet

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

PLATE NO.
TP 16

## GREAT VALLEY COMPLEX

**KJgv** — **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.
*Primarily present in site 3C-2 as sandstone conglomerate, shale, meta shale & graywacke sequences*

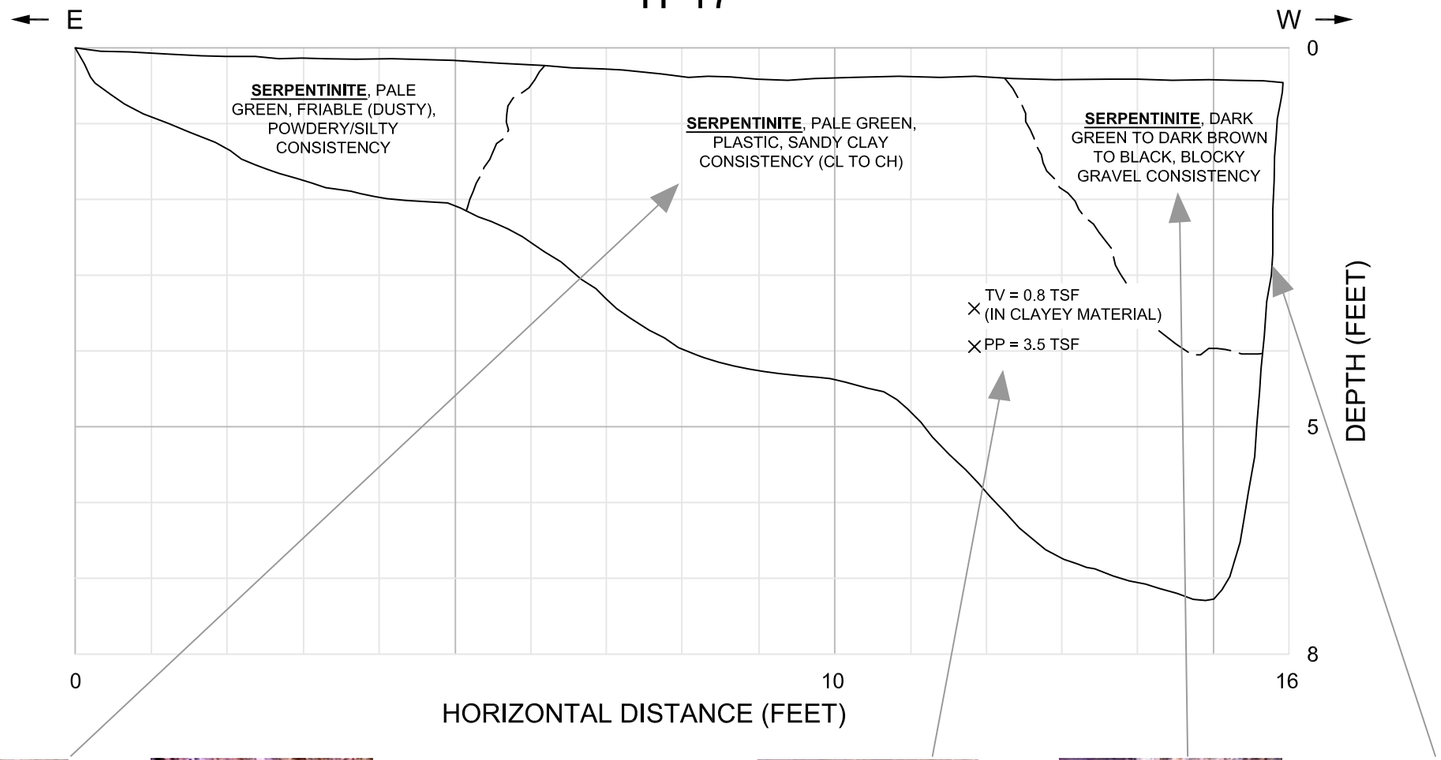
**Jsv** — **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).
*Primarily present in site 3C-2 as isolated surface and near surface exotic blocks of intermediate tuff & meta tuff. Gray-green, very strong*

## COAST RANGE OPHIOLITE

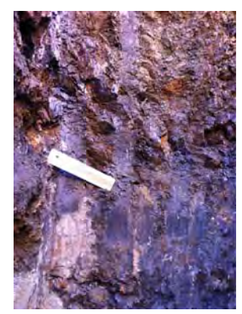
**spm** — **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.
*Primarily present in site 3C-2 as highly variable sheared serpentinite*

PP POCKET PENETROMETER READING
TV TORVANE READING

# TP17

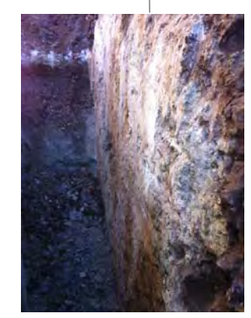


TP-17A


TP-17B

TP-17C

TP-17D


TP-17E

TP-17F

| | | | |
|---|---|---|---|
| TEST PIT ID | TP17 | ORIENTATION | N55E |
| DATE EXCAVATED | 11/14/12 | WIDTH | 2 FEET |
| DATE BACKFILLED | 11/14/12 | MAX DEPTH | 7.5 FEET |
| LOGGED FACE | SE WALL | LENGTH | 16 FEET |

COMMENTS
- HIGHLY VARIABLE SHEARED SERPENTINITE, (spm)
- NO GROUNDWATER ENCOUNTERED
- WHEELROLLED TO EXISTING GROUND SURFACE

COMPILED BY: N. SCHEEMAKER
REVISED BY: N. SCHEEMAKER
CHECKED BY: A. JOHNS
DATE: 01/20/2013
APPROVED BY: Y. PRASHAR

# TEST PIT 17

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / GEO 2
SCALE: 1 inch:2 feet

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

PLATE NO.
TP 17

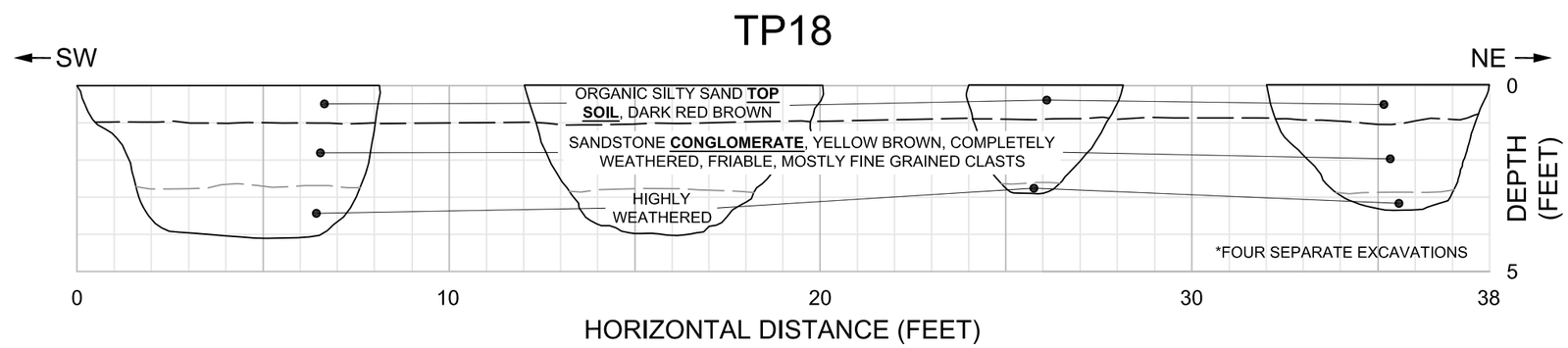
## GREAT VALLEY COMPLEX

**KJgv** — **Sandstone, shale, and Conglomerate (Early Cretaceous and Late Jurassic)** - Predominantly pebble to cobble conglomerate, but also includes distinctly bedded, dark- to light-gray, brown-weathering, coarse-grained lithic wacke and shale. Clasts include quartz- and plagioclase-porphyry volcanics, black chert and quartzite.
*Primarily present in site 3C-2 as sandstone conglomerate, shale, meta shale & graywacke sequences*

**Jsv** — **Keratophyre and quartz keratophyre tuff (Late Jurassic)** - Well-bedded, white to red- and green-stained, intermediate and silicic tuff, and greenish-gray fine- to coarse-grained tuffaceous sandstone. Contains Late Jurassic (Tithonian) radiolarians, and thought to be correlative with tuff of the Lotta Creek Formation of the western San Joaquin Valley (Blake and others, 2002).
*Primarily present in site 3C-2 as isolated surface and near surface exotic blocks of intermediate tuff & meta tuff. Gray-green, very strong*

## COAST RANGE OPHIOLITE

**spm** — **Serpentinite matrix melange (late and middle Jurassic)** - Sheared meta-serpentinite containing blocks and lenses of greenstone, high-grade metamorphic rocks, and graywacke.
*Primarily present in site 3C-2 as highly variable sheared serpentinite*




| | | | |
|---|---|---|---|
| TEST PIT ID | TP18 | ORIENTATION | N33E |
| DATE EXCAVATED | 11/14/12 | WIDTH | 2 FEET |
| DATE BACKFILLED | 11/14/12 | MAX DEPTH | 4 FEET |
| LOGGED FACE | S WALL | LENGTH | 38 FEET |

COMMENTS
- SANDSTONE CONGLOMERATE BEDROCK, (KJgv)
- NO GROUNDWATER ENCOUNTERED
- WHEELROLLED TO EXISTING GROUND SURFACE

| | |
|---|---|
| COMPILED BY: N. SCHEEMAKER | |
| REVISED BY: N. SCHEEMAKER | |
| CHECKED BY: A. JOHNS | |
| DATE: 01/20/2013 | APPROVED BY: Y. PRASHAR |

# TEST PIT 18

OCSD WASTEWATER STORAGE AND RECLAMATION PROJECT
OCCIDENTAL, CALIFORNIA

PROJECT NO: 01202 / GEO 2

SCALE: 1 inch:4 feet

M2 CONSULTANTS inc
638 19th STREET, UNIT 18
SAN FRANCISCO, CALIFORNIA, 94107

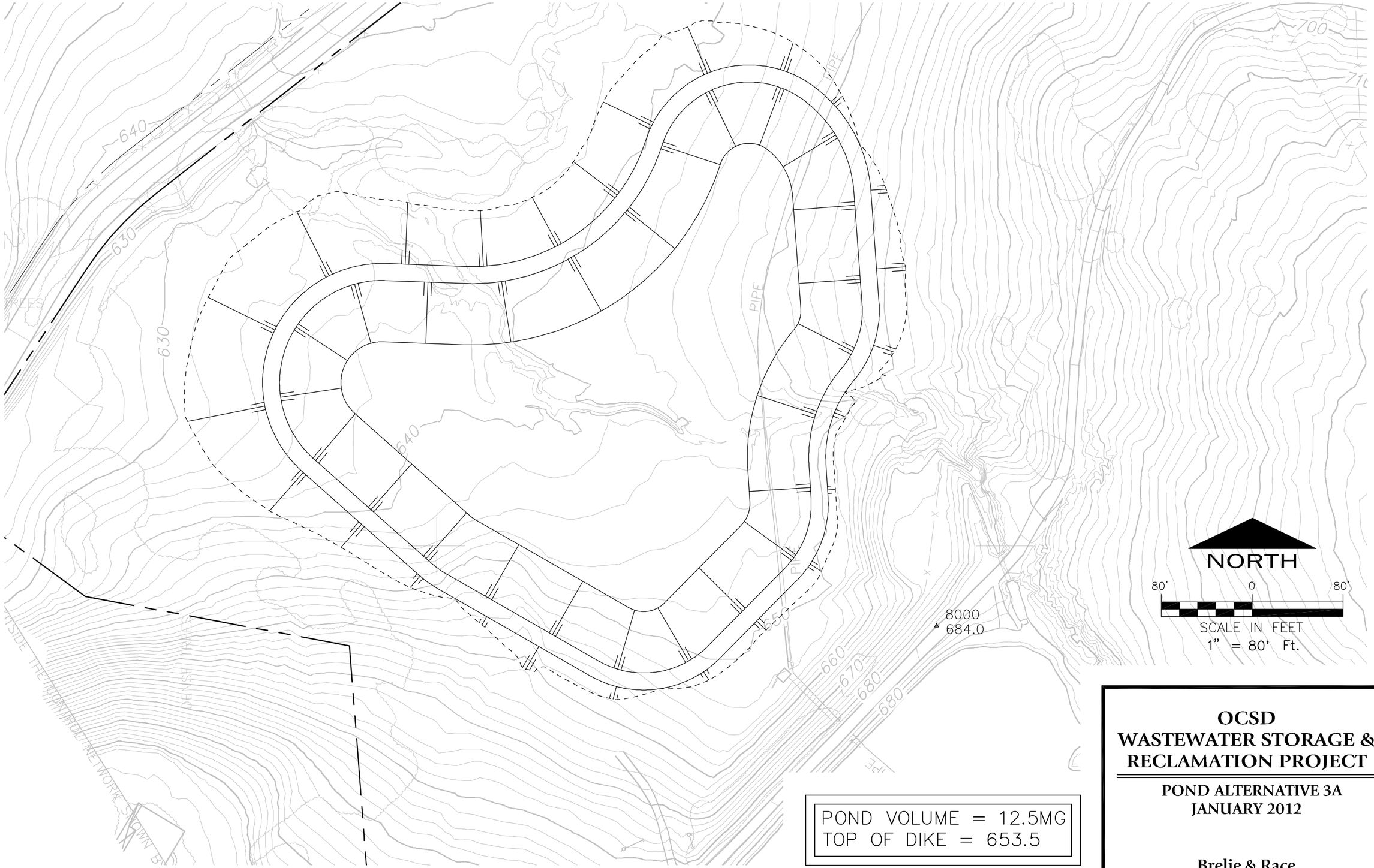
PLATE NO.
TP 18

# APPENDIX D

## FIGURES AND DATA FOR POND OPTIONS DEVELOPED DURING SITE REVIEW

OCSD Wastewater Storage and Reclamation Project
Pond Volumes and Earthwork Volumes

| Pond | Top Inside Hinge Elevation | Water Surface Elevation | Depth of Water | Bottom of Pond | Maximum Dike Height | Volume (MG) | Cut Volume (Yd^3) (unadjusted) | Fill Volume (Yd^3) (unadjusted) | Net Earthwork (yd^3) |
|---|---|---|---|---|---|---|---|---|---|
| 3A | 653.5 | 650.5 | 15 | 635.5 | 23.1 | 12.5 | 33730 | 30,417 | 3,313 |
| | | | | | | | | | |
| 3B | 676 | 673 | 15 | 658 | 22.2 | 12.5 | 36892 | 27,077 | 9,815 |
| | | | | | | | | | |
| 3C | 870 | 867 | 24 | 843 | 24.9 | 12.5 | 100445 | 21,485 | 78,960 |
| | | | | | | | | | |
| 3D | 788 | 785 | 15 | 770 | 24.2 | 1.9 | 18471 | 8,340 | 10,131 |
| | | | | | | | | | |
| 3E | 675 | 672 | 18.5 | 653.5 | 24.8 | 12.5 | 62852 | 30,890 | 31,962 |



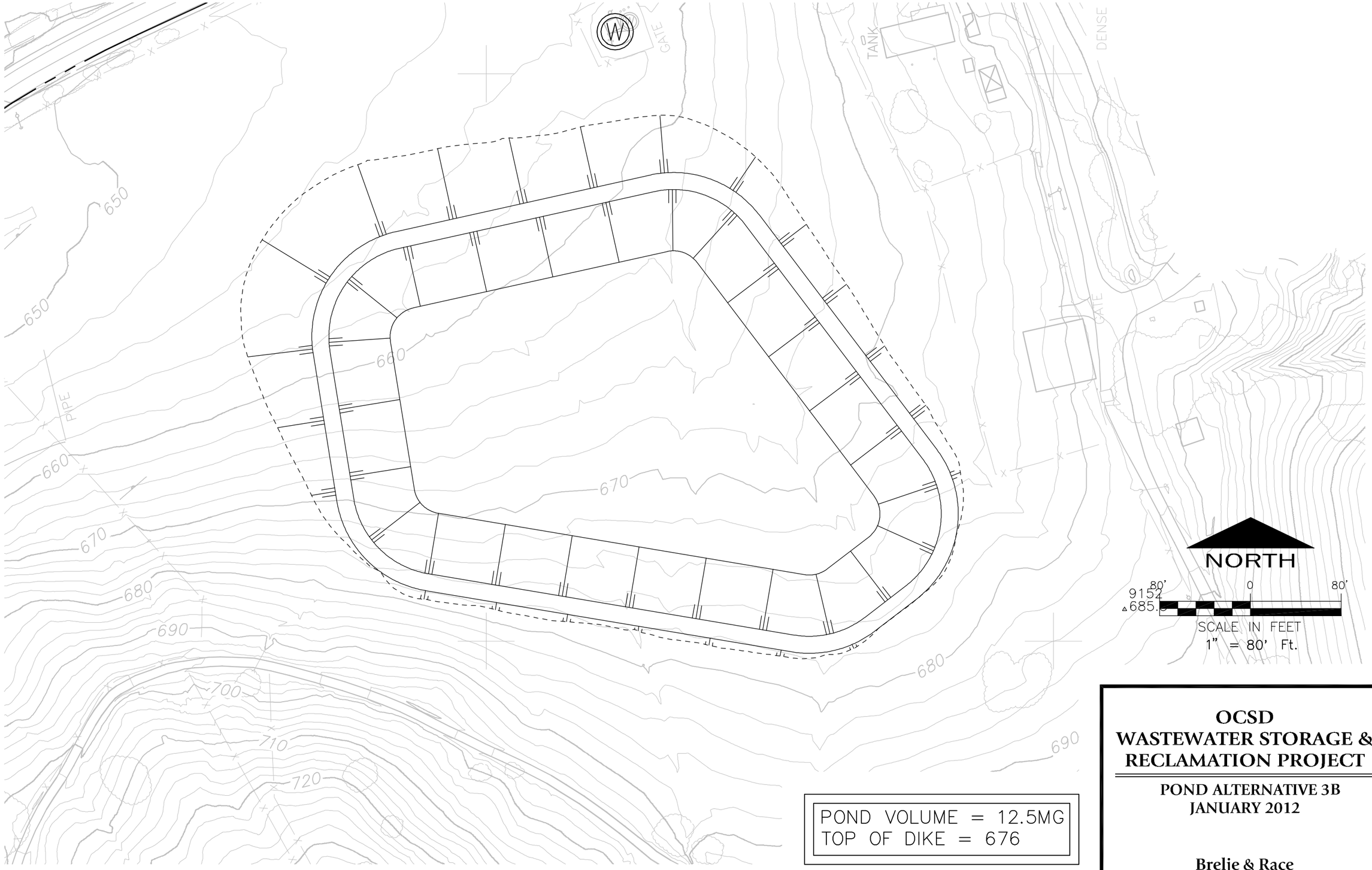

# OCSD
# WASTEWATER STORAGE & RECLAMATION PROJECT

## POND ALTERNATIVE 3A
## JANUARY 2012

Brelje & Race
CONSULTING CIVIL ENGINEERS

01-31-12 bezazor \3619\dwg\3619 01\3619.01-BASE.dwg TAB: A4-(2)

APPENDIX D - FIGURE 1



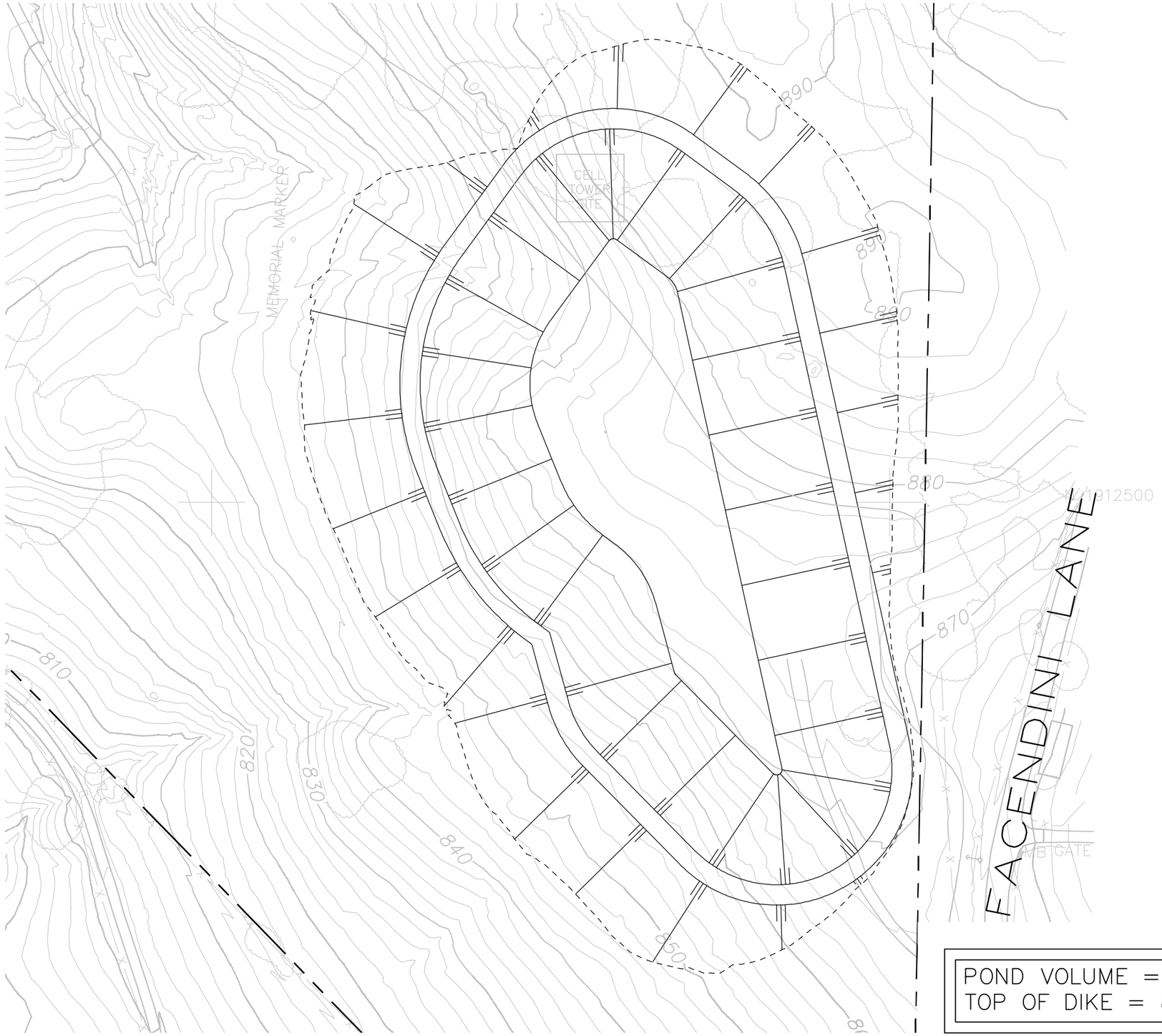

# OCSD
# WASTEWATER STORAGE & RECLAMATION PROJECT

## POND ALTERNATIVE 3B
## JANUARY 2012

Brelje & Race
CONSULTING CIVIL ENGINEERS

01-31-12 beazor \3619\dwg\3619 01\3619.01-BASE.dwg TAB: B2-(3)

APPENDIX D - FIGURE 2




NORTH

80' 0 80'

SCALE IN FEET
1" = 80' Ft.

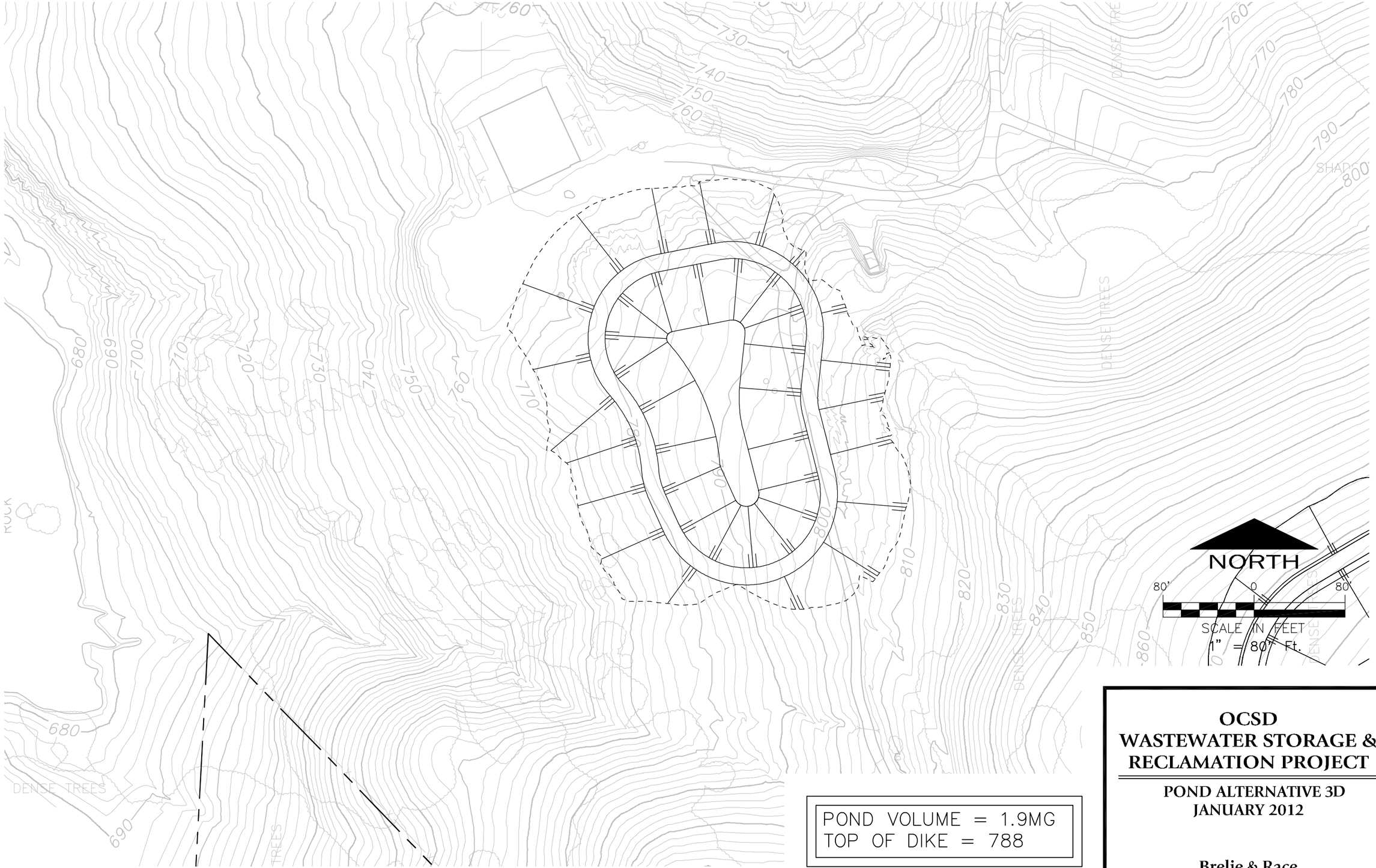
POND VOLUME = 12.5MG
TOP OF DIKE = 870

**OCSD**
**WASTEWATER STORAGE & RECLAMATION PROJECT**

**POND ALTERNATIVE 3C**
**JANUARY 2012**

Brelje & Race
CONSULTING CIVIL ENGINEERS

01-31-12 beazer \3619\dwg\3619 01\3619.01-BASE.dwg TAB: C5-(4)



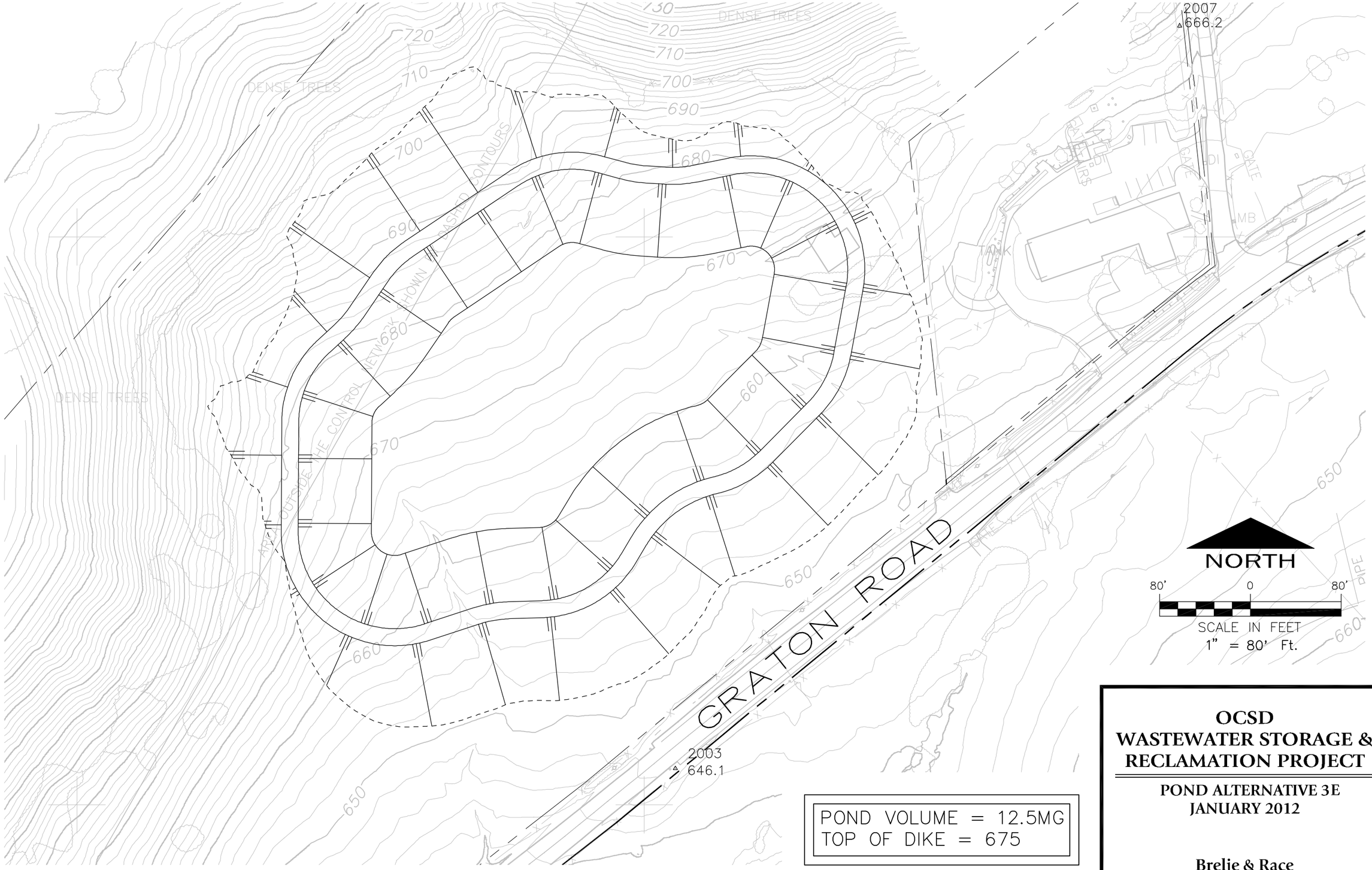

# OCSD
# WASTEWATER STORAGE & RECLAMATION PROJECT

## POND ALTERNATIVE 3D
## JANUARY 2012

Brelje & Race
CONSULTING CIVIL ENGINEERS

01-31-12 bezzor \3619\dwg\3619 01\3619.01-BASE.dwg TAB: 02-(5)

APPENDIX D - FIGURE 4




**OCSD**
**WASTEWATER STORAGE & RECLAMATION PROJECT**

**POND ALTERNATIVE 3E**
**JANUARY 2012**

Brelje & Race
CONSULTING CIVIL ENGINEERS

01-31-12 benzor \3619\dwg\3619 01\3619.01-BASE.dwg TAB: E3-(6)

APPENDIX D - FIGURE 5

# APPENDIX E

## LABORATORY REPORTS – ASBESTOS IN SITE ROCKS AND SOILS

## Richard Ingram

**From:** Richard Ingram
**Sent:** Friday, March 02, 2012 9:36 AM
**To:** Jim Flores
**Cc:** Mary Hiatt
**Subject:** FW: Rock Samples for Asbestos Analysis - Occidental Site

Jim:

The samples will be labeled as follows:

Test Pit 1
Test Pit 2
Test Pit 3
Test Pit 4
Test Pit 5
Test Pit 6
Sample 7 – East Side 50 foot South of Grahams Pond Overflow Culvert
Sample 8 – In cut of meander, Alluvial at 5 feet deep, east side of creek bank , 75 feet south of boring no. B4

Thanks,

**Richard J. Ingram**
Vice President
**Brelje & Race Consulting Engineers**
v. (707) 576-1322
f. (707) 576-0469
www.brce.com

***...where excellence, innovation and integrity work for you every day.***

---

**From:** Richard Ingram
**Sent:** Friday, March 02, 2012 9:13 AM
**To:** 'Jim Flores'
**Cc:** Mary Hiatt
**Subject:** Rock Samples for Asbestos Analysis - Occidental Site

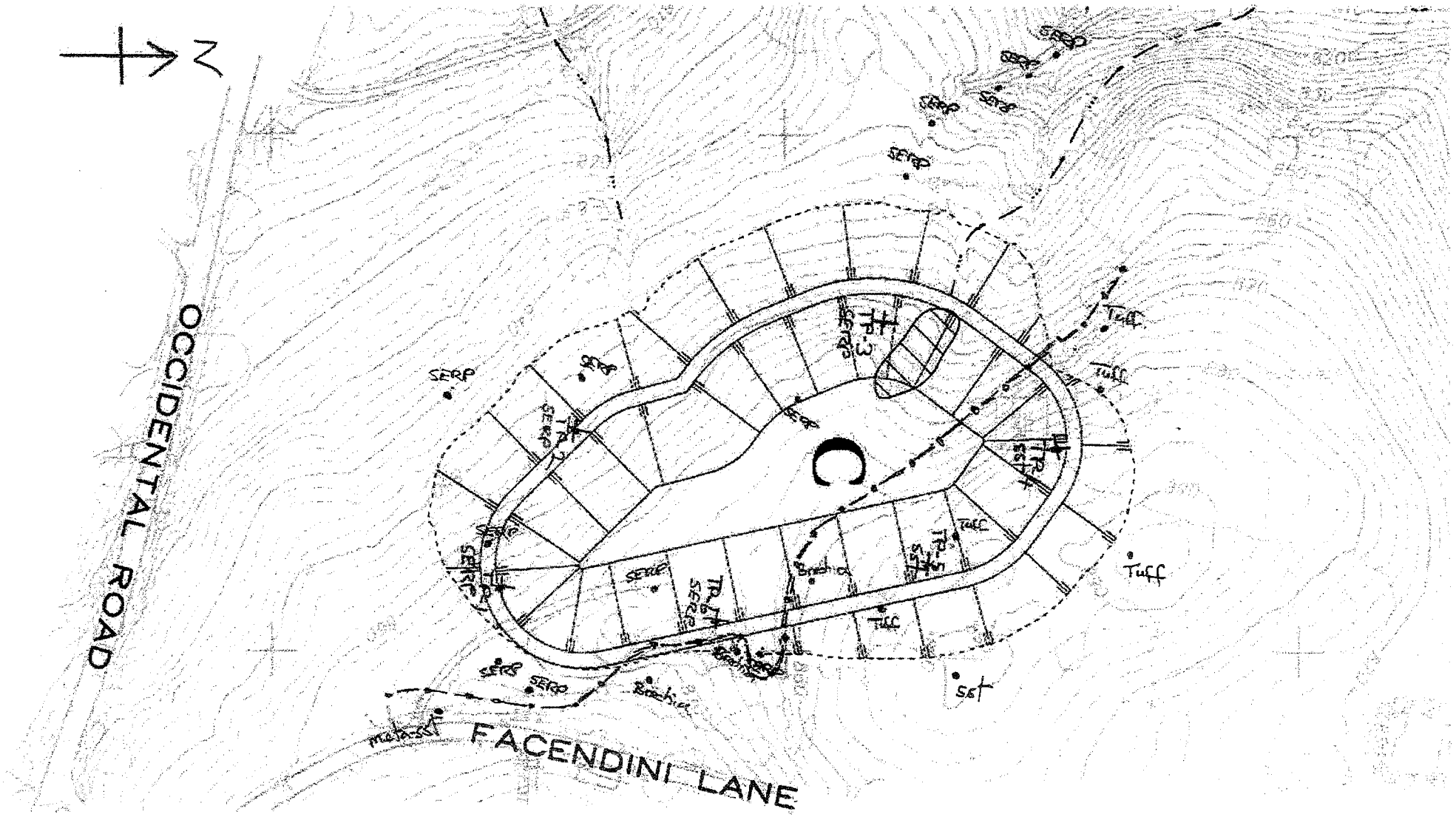
Jim:

We will be dropping off 8 samples for asbestos analysis today before 5pm.

We would like each sample to be analyzed to determine % asbestos per the ARB test method 435.

We understand that cost be sample for the analysis would be $85.00 and the schedule for results would be Friday, March 9, 2012.

Thank you,




KEY

TP-1: Test Pit

• Surface Bedrock Outcrop with Rock Type

Appox. Formation Boundary Based on Test Pits & Surface Bedrock Outcrops.

Forensic Analytical Laboratories

**Final Report**

# Bulk Asbestos Material Analysis

(Air Resources Board Method 435, June 6, 1991)

Brelje & Race Consulting Engineers
Richard Ingram
475 Aviation Blvd., Ste 120

Santa Rosa, CA 95403

**Client ID:** CH1000
**Report Number:** N004377
**Date Received:** 03/02/12
**Date Analyzed:** 03/09/12
**Date Printed:** 03/09/12

**Job ID/Site:** Occidental Site

**PLM Report Number:** N/A

**FALI Job ID:** CH1000-4071
**Total Samples Submitted:** 8
**Total Samples Analyzed:** 8

**Sample Preparation and Analysis:**

Samples were analyzed by the Air Resources Board's Method 435, Determination of Asbestos Content of Serpentine Aggregate. Samples were ground to 200 particle size in the laboratory. Approximately 1 pint was retained for analysis. Samples were prepared for observation according to the guidelines of Exception I and Exception II as defined by the 435 Method. Samples which contained less than 10% asbestos were prepared for observation according to the point count technique as defined by the 435 Method. This analysis was performed with a standard cross-hair reticle.

| Sample ID | Lab Number | Layer Description |
|---|---|---|
| **Test pit 1** | 11226854 | **Grey/Green Soil** |

*Point Count Results:*

| | |
|---|---|
| Number of asbestos points counted: | 9 |
| Number of non-empty points: | 400 |
| Matrix percentage of entire sample: | 100 |
| Percent asbestos in matrix: | 2.3 |
| Visual estimation percentage: | 2.0 |
| Asbestos type(s) detected: | Chrysotile |

Comment:

| | | |
|---|---|---|
| **Test pit 2** | 11226855 | **Grey/Green Soil** |

*Point Count Results:*

| | |
|---|---|
| Number of asbestos points counted: | 3 |
| Number of non-empty points: | 400 |
| Matrix percentage of entire sample: | 100 |
| Percent asbestos in matrix: | 0.75 |
| Visual estimation percentage: | Trace |
| Asbestos type(s) detected: | Chrysotile |

Comment:



3777 Depot Road, Suite 409, Hayward, CA 94545 / Telephone: (510) 887-8828 (800) 827-FASI / Fax: (510) 887-4218

Forensic Analytical Laboratories

# Bulk Asbestos Material Analysis

(Air Resources Board Method 435, June 6, 1991)

Brelje & Race Consulting Engineers
Richard Ingram
475 Aviation Blvd., Ste 120

Santa Rosa, CA 95403

| | |
|---|---|
| **Client ID:** | CH1000 |
| **Report Number:** | N004377 |
| **Date Received:** | 03/02/12 |
| **Date Analyzed:** | 03/09/12 |
| **Date Printed:** | 03/09/12 |

**Job ID/Site:** Occidental Site

**PLM Report Number:** N/A

| | |
|---|---|
| **FALI Job ID:** | CH1000-4071 |
| **Total Samples Submitted:** | 8 |
| **Total Samples Analyzed:** | 8 |

**Sample Preparation and Analysis:**

Samples were analyzed by the Air Resources Board's Method 435, Determination of Asbestos Content of Serpentine Aggregate. Samples were ground to 200 particle size in the laboratory. Approximately 1 pint was retained for analysis. Samples were prepared for observation according to the guidelines of Exception I and Exception II as defined by the 435 Method. Samples which contained less than 10% asbestos were prepared for observation according to the point count technique as defined by the 435 Method. This analysis was performed with a standard cross-hair reticle.

| Sample ID | Lab Number | Layer Description |
|---|---|---|
| **Test pit 3** | 11226856 | **Grey/Green Soil** |

*Point Count Results:*

| | |
|---|---|
| Number of asbestos points counted: | 1 |
| Number of non-empty points: | 400 |
| Matrix percentage of entire sample: | 100 |
| Percent asbestos in matrix: | 0.25 |
| Visual estimation percentage: | Trace |
| Asbestos type(s) detected: | Chrysotile |

Comment:

**Test pit 4** 11226857 **Brown Soil**

*Visual Estimation Results:*

| | |
|---|---|
| Matrix percentage of entire sample: | 100 |
| Visual estimation percentage: | None Detected |
| Asbestos type(s) detected: | None Detected |

Comment: This result meets the requirements of Exception I as defined by the 435 Method.

**Test pit 5** 11226858 **Brown Soil**

*Visual Estimation Results:*

| | |
|---|---|
| Matrix percentage of entire sample: | 100 |
| Visual estimation percentage: | None Detected |
| Asbestos type(s) detected: | None Detected |

Comment: This result meets the requirements of Exception I as defined by the 435 Method.



Hayward, CA 94545 / Telephone: (510) 887-8828 (800) 827-FASI / Fax: (510) 887-4218

# Bulk Asbestos Material Analysis

(Air Resources Board Method 435, June 6, 1991)

Brelje & Race Consulting Engineers
Richard Ingram
475 Aviation Blvd., Ste 120

Santa Rosa, CA 95403

**Client ID:** CH1000
**Report Number:** N004377
**Date Received:** 03/02/12
**Date Analyzed:** 03/09/12
**Date Printed:** 03/09/12

**Job ID/Site:** Occidental Site

**FALI Job ID:** CH1000-4071
**Total Samples Submitted:** 8
**Total Samples Analyzed:** 8

**PLM Report Number:** N/A

**Sample Preparation and Analysis:**

Samples were analyzed by the Air Resources Board's Method 435, Determination of Asbestos Content of Serpentine Aggregate. Samples were ground to 200 particle size in the laboratory. Approximately 1 pint was retained for analysis. Samples were prepared for observation according to the guidelines of Exception I and Exception II as defined by the 435 Method. Samples which contained less than 10% asbestos were prepared for observation according to the point count technique as defined by the 435 Method. This analysis was performed with a standard cross-hair reticle.

| Sample ID | Lab Number | Layer Description |
|---|---|---|
| **Test pit 6** | 11226859 | **Grey/Green Soil** |

*Point Count Results:*

| | |
|---|---|
| Number of asbestos points counted: | 9 |
| Number of non-empty points: | 400 |
| Matrix percentage of entire sample: | 100 |
| Percent asbestos in matrix: | 2.3 |
| Visual estimation percentage: | 2.0 |
| Asbestos type(s) detected: | Chrysotile |

Comment:

| | | |
|---|---|---|
| **Sample 7** | 11226860 | **Grey/Green Soil** |

*Point Count Results:*

| | |
|---|---|
| Number of asbestos points counted: | 10 |
| Number of non-empty points: | 400 |
| Matrix percentage of entire sample: | 100 |
| Percent asbestos in matrix: | 2.5 |
| Visual estimation percentage: | 2.0 |
| Asbestos type(s) detected: | Chrysotile |

Comment:



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Forensic Analytical Laboratories

**Final Report**

# Bulk Asbestos Material Analysis

(Air Resources Board Method 435, June 6, 1991)

Brelje & Race Consulting Engineers
Richard Ingram
475 Aviation Blvd., Ste 120

Santa Rosa, CA 95403

**Client ID:** CH1000
**Report Number:** N004377
**Date Received:** 03/02/12
**Date Analyzed:** 03/09/12
**Date Printed:** 03/09/12

**Job ID/Site:** Occidental Site

**PLM Report Number:** N/A

**FALI Job ID:** CH1000-4071
**Total Samples Submitted:** 8
**Total Samples Analyzed:** 8

**Sample Preparation and Analysis:**

Samples were analyzed by the Air Resources Board's Method 435, Determination of Asbestos Content of Serpentine Aggregate. Samples were ground to 200 particle size in the laboratory. Approximately 1 pint was retained for analysis. Samples were prepared for observation according to the guidelines of Exception I and Exception II as defined by the 435 Method. Samples which contained less than 10% asbestos were prepared for observation according to the point count technique as defined by the 435 Method. This analysis was performed with a standard cross-hair reticle.

| Sample ID | Lab Number | Layer Description |
|---|---|---|
| **Sample 8** | 11226861 | **Brown Soil** |

*Point Count Results:*

| | |
|---|---|
| Number of asbestos points counted: | 3 |
| Number of non-empty points: | 400 |
| Matrix percentage of entire sample: | 100 |
| Percent asbestos in matrix: | 0.75 |
| Visual estimation percentage: | Trace |
| Asbestos type(s) detected: | Chrysotile |

Comment:

Tad Thrower

Tad Thrower, Laboratory Supervisor, Hayward Laboratory

Note: Limit of Quantification (LOQ) = 0.25%. Trace denotes the presence of asbestos below the LOQ. ND = None Detected.

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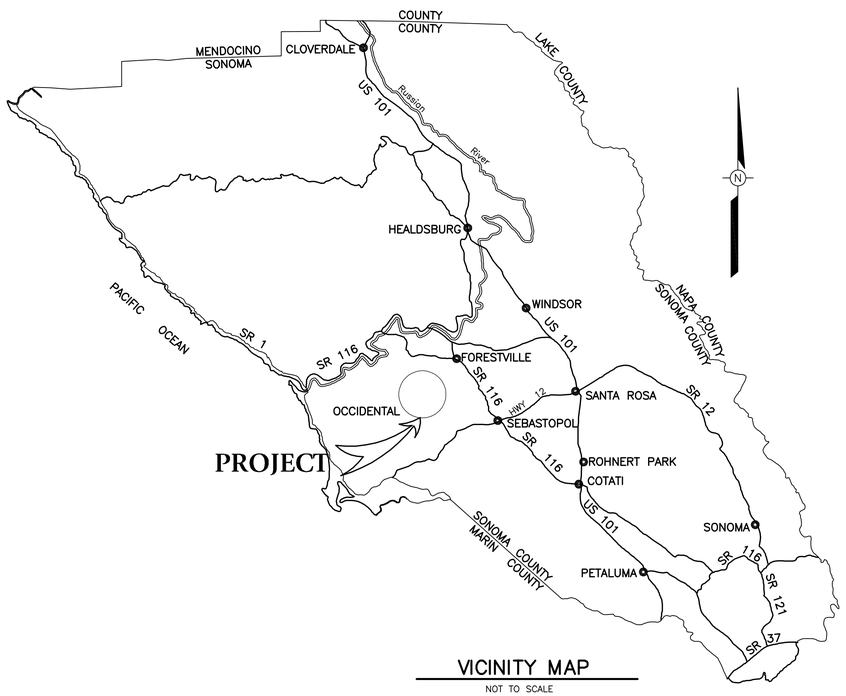
# APPENDIX F

## 30% CONCEPTUAL DESIGN DRAWINGS FOR THE RECOMMENDED PROJECT

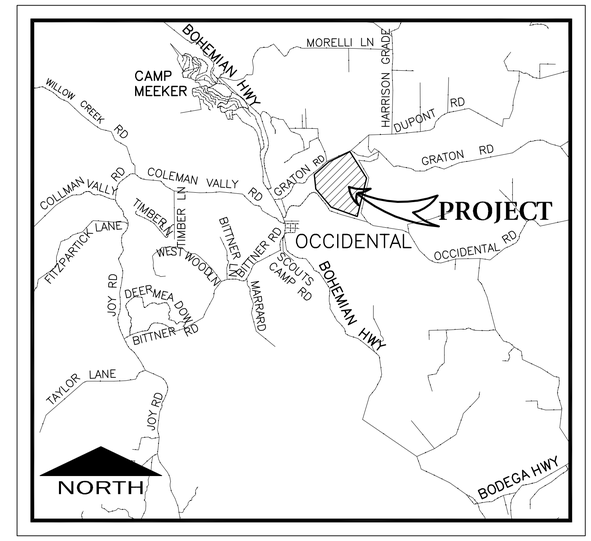
# OCCIDENTAL COUNTY SANITATION DISTRICT

# WASTEWATER STORAGE AND RECLAMATION PROJECT




VICINITY MAP

NOT TO SCALE




LOCATION MAP

SCALE : 1" = 3500'

## INDEX OF SHEETS


| SHEET NO. | DRAWING NO. | SHEET TITLE |
|---|---|---|
| 1 | G-1 | LOCATION & VICINITY MAPS & INDEX TO DRAWINGS |
| 2 | G-2 | ABBREVIATIONS, LEGEND, NOTES & TYPICAL DETAILS |
| 3 | C-1 | DESIGN CRITERIA, SCHEMATIC PLAN & PROFILE |
| 4 | C-2 | OVERALL SITE PLAN & KEY MAP |
| 5 | C-3 | TRANSFER PUMP STATION PLAN, SECTIONS & DETAILS |
| 6 | C-4 | TRANSFER MAIN & IRRIGATION PLAN |
| 7 | C-5 | TRANSFER MAIN & IRRIGATION PLAN |
| 8 | C-6 | TRANSFER MAIN & IRRIGATION PLAN |
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| 12 | C-10 | EFFLUENT STORAGE POND GRADING PLAN |
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| 15 | C-13 | EFFLUENT STORAGE POND DETAILS |
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SONOMA COUNTY WATER AGENCY

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LOCATION & VICINITY MAPS
& INDEX TO DRAWINGS

FILE NAME: 3619.06 COVER.dwg
CONTRACT NUMBER: 7404
DRAWING NUMBER: G-1
SHEET 1 OF XX

02-14-13 kifi:zghi \3619\dwg\3619.06\7404-COVER-3619.06 COVER.dwg TAB: G-1

## ABBREVIATIONS

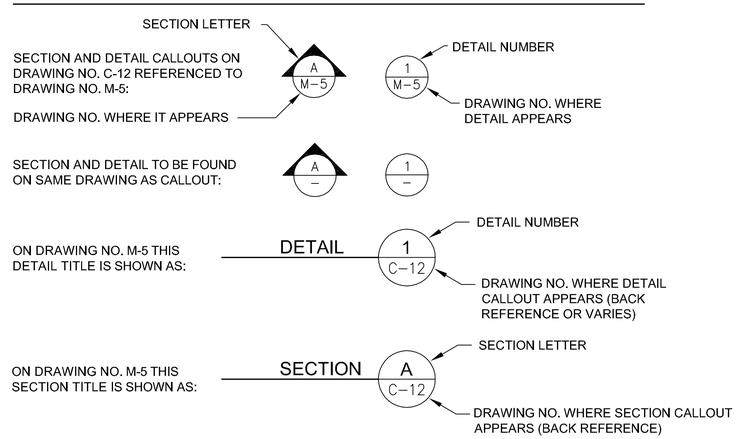
| | |
|---|---|
| AB | AGGREGATE BASE |
| AC | ASPHALT CONCRETE |
| ACP | ASPHALT CEMENT PIPE |
| AD | AREA DRAIN |
| APPROX | APPROXIMATE |
| BLD | BUILDING |
| BO | BLOWOFF |
| CIP | CAST-IN-PLACE |
| CI | CURB INLET |
| CLSM | CONTROLLED LOW STRENGTH MATERIAL |
| CMP | CORRUGATED METAL PIPE |
| CP | CONCRETE PIPE |
| CL | CLASS |
| CSR | CITY OF SANTA ROSA |
| CU FT | CUBIC FOOT |
| CU YD | CUBIC YARD |
| DI | DROP INLET |
| DIP | DUCTILE IRON PIPE |
| DIA | DIAMETER |
| D/W | DRIVEWAY |
| DWG | DRAWING |
| E | EAST |
| EA | EACH |
| EA WY | EACH WAY |
| EG | EXISTING GRADE |
| EL | ELEVATION |
| ESMT | EASEMENT |
| EX | EXISTING |
| EP | EDGE OF PAVEMENT |
| FG | FINISHED GRADE |
| FH | FIRE HYDRANT |
| FL | FLOWLINE |
| FLG | FLANGE |
| FNC | FENCE |
| FOC | FACE OF CURB |
| HDPE | HIGH-DENSITY POLYETHYLENE |
| HWY | HIGHWAY |
| HP | HORSE POWER |
| ID | INSIDE DIAMETER |
| INV | INVERT |
| IRR | IRRIGATION |
| JP | JOINT POLE |
| L | LENGTH |
| LAT | LATERAL |
| LF | LINEAR FEET |
| LT | LEFT |
| MAX | MAXIMUM |
| MB | MAILBOX |
| MIL | MILLIMETER |
| MIN | MINIMUM |
| MSL | MEAN SEA LEVEL |
| N | NORTH |
| NO | NUMBER |
| OD | OUTSIDE DIAMETER |
| PCC | PORTLAND CEMENT CONCRETE |
| PERF | PERFORATED |
| PSI | POUNDS PER SQUARE INCH |
| PSSE | PUBLIC SANITARY SEWER EASEMENT |
| PUE | PUBLIC UTILITY EASEMENT |
| PVC | POLYVINYL CHLORIDE |
| RC | RELATIVE COMPACTION |
| RCB | REINFORCED CONCRETE BOX |
| RCP | REINFORCED CONCRETE PIPE |
| RDWD | REDWOOD |
| RT | RIGHT |
| S | SLOPE/SOUTH |
| SCWA | SONOMA COUNTY WATER AGENCY |
| SD | STORM DRAIN |
| S/L | STREET LIGHT |
| SS | SANITARY SEWER OR STAINLESS STEEL |
| SSCO | SANITARY SEWER CLEANOUT |
| SSMH | SANITARY SEWER MANHOLE |
| STA | STATION |
| STD | STANDARD |
| TCE | TEMPORARY CONSTRUCTION EASEMENT |
| TYP | TYPICAL |
| UG | UNDER GROUND |
| VCP | VITRIFIED CLAY PIPE |
| W | WEST |
| W/ | WITH |
| WD | WOOD |
| XING | CROSSING |
| +/- | PLUS/MINUS |

## LEGEND

- ABANDON EXIST SS MAIN
- PLUG EXIST SS MAIN AT SSMH
- CONNECTION TO EXISTING LATERAL
- SANITARY SEWER MANHOLE - NEW
- SANITARY SEWER MANHOLE - EXISTING
- SANITARY SEWER MANHOLE - REMOVE AND REPLACE
- SANITARY SEWER CLEANOUT - NEW
- SANITARY SEWER CLEANOUT - EXISTING
- PROJECT IDENTIFICATION SIGN
- FIRE HYDRANT
- CENTERLINE MONUMENT/PROPERTY CORNER
- WV — WATER VALVE EXISTING
- WV — WATER VALVE NEW
- XXXX — PARCEL ADDRESS
- SIGN
- MB — MAIL BOX
- UTILITY POLE
- GUY ANCHOR
- UTILITY VAULT

## GENERAL NOTES

## TYPICAL SECTION LETTERING AND DETAIL NUMBERING SYSTEM

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ABBREVIATIONS, LEGEND, NOTES & TYPICAL DETAILS

FILE NAME: 3619.06 NOTE.dwg
CONTRACT NUMBER: 7404
DRAWING NUMBER: G-2
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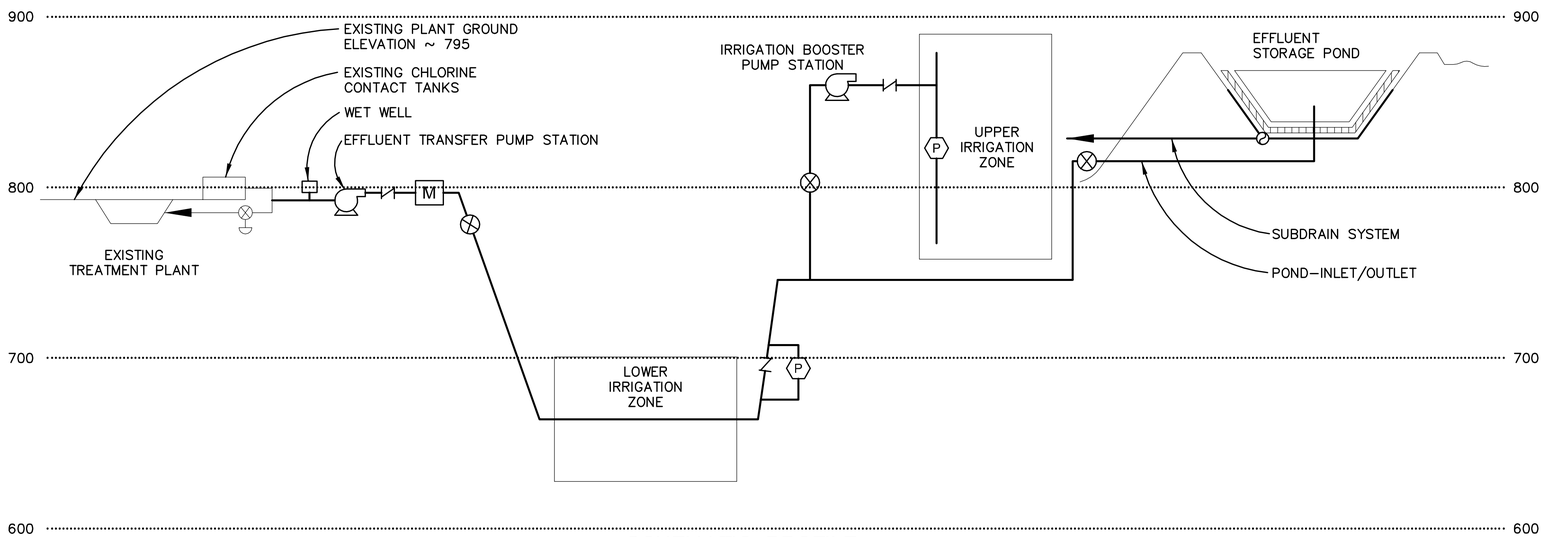
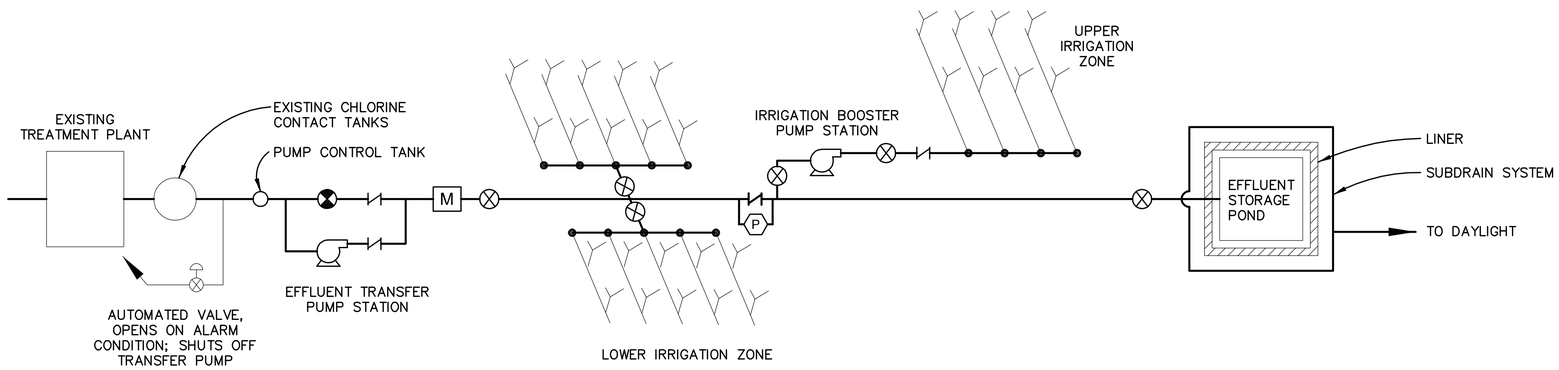



## SCHEMATIC PROFILE

NOT TO SCALE




## SCHEMATIC PLAN

NOT TO SCALE

## LEGEND

| SYMBOL | DESCRIPTION |
|---|---|
| ⊗ | MANUAL VALVE – NORMALLY OPEN |
| ⧗ | MANUAL VALVE – NORMALLY CLOSED |
| ⊗ | AUTOMATICALLY OPERATED VALVE |
| ⋈ | CHECK VALVE |
| M | FLOW METER |
| | PUMP |
| | SPRINKLER |
| | IRRIGATION VALVE |
| P | PRESSURE REDUCING VALVE AND BYPASS WITH CHECK VALVE |

## DESIGN CRITERIA

### Storage Pond

| | |
|---|---|
| Volume | 12.5 Million Gallons |
| Elevations | |
| Top of Dike | 880.0 Feet MSL |
| Bottom | 840.5 Feet MSL |
| Max. Water Surface | 877.2 Feet MSL |
| Max. Storage | 887.0 Feet MSL |
| Maximum Fill Rate | 160 Gallons per Minute |

### Irrigation System

| | |
|---|---|
| Irrigation Areas | |
| Served by Pond 3C–2 (Lower Irrigation Zone) | 17.9 Acres |
| Served by Booster Pump (Upper Irrigation Zone) | 6.0 Acres |
| Elevation Range | |
| Areas Served by Pond 3C–2 | 630 – 690 Feet MSL |
| Areas Served by Booster Pump | 760 – 890 Feet MSL |
| Irrigation Booster Pump Station | |
| Flow | 145 Gallons per Minute |
| Horsepower | 7.5 HP |
| Total Dynamic Head | 145 Feet |

### Effluent Transfer

| | |
|---|---|
| Transfer Pump Station | |
| Flow | 155 Gallons per Minute |
| Horsepower | 7.5 HP |
| Total Dynamic Head | 95 Feet |
| Pump Control Tank | 1,000 Gallons |

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OCCIDENTAL COUNTY SANITATION DISTRICT
DESIGN CRITERIA, SCHEMATIC PLAN & PROFILE

FILE NAME: 3619.06 SCHEMATIC.dwg
CONTRACT NUMBER: 7404
DRAWING NUMBER: C-1
SHEET 3 OF XX

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02-14-13

## LEGEND

- CREEK OR SURFACE DRAINAGE
- LOWER IRRIGATION AREA 17.9 ACRES
- UPPER IRRIGATION AREA 6.2 ACRES
- 6" ABOVE GROUND IRRIGATION MAIN WITH 4" IRRIGATION VALVES
- 6" BURIED TRANSMISSION MAIN

## NOTES

1. IRRIGATION VALVES SPACED AT 60 FOOT INTERVALS.
2. SURFACE IRRIGATION – 2" AND 3" ALUMINUM LATCH PIPE WITH 3/4" DIAMETER IMPACT SPRINKLERS, 40' X 60' SPACING, 8–10 GPM / SPRINKLER, ORIFICE DIAMETER RANGE 3/16" – 13/64"

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OVERALL SITE PLAN & KEY MAP

FILE NAME: 3619.06 BASE.dwg
CONTRACT NUMBER: 7404
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SHEET 4 OF XX

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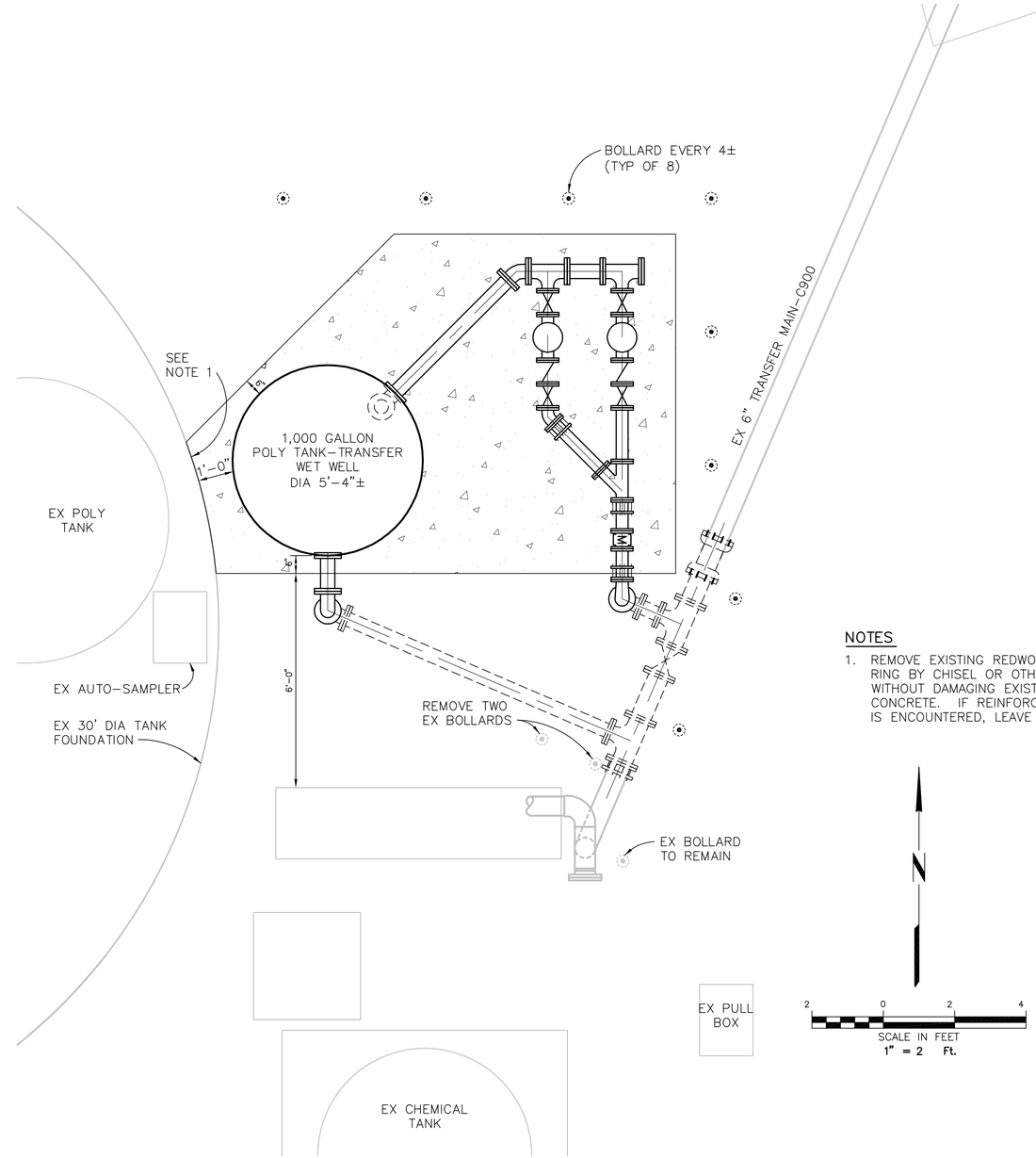


PLAN

NOTES

1. REMOVE EXISTING REDWOOD CROZE RING BY CHISEL OR OTHER MEANS, WITHOUT DAMAGING EXISTING CONCRETE. IF REINFORCING BAR IS ENCOUNTERED, LEAVE IN PLACE.

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TRANSFER PUMP STATION PLAN, SECTIONS & DETAILS

FILE NAME: 3619.06 BASE.dwg
CONTRACT NUMBER: 7404
DRAWING NUMBER: C-3
SHEET 5 OF XX

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TRANSFER MAIN & IRRIGATION PLAN

FILE NAME: 3619.06 BASE.dwg
CONTRACT NUMBER: 7404
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GRATON ROAD

6" TRANSFER MAIN (BURIED)

6" IRR MAIN

6" VALVE (BURIED)

IRR RISER WITH 4" VALVE

6" IRR MAIN WITH 4" IRR VALVES-ABOVE GROUND ALUMINUM (TYP)

6" TRANSFER MAIN (BURIED)

6" TRANSFER MAIN

IRR RISER WITH 4" VALVE

6" IRR MAIN

6" IRR MAIN

9+00 10+00 11+00 12+00 13+00 14+00 15+00 16+00 17+00 18+00 19+00 20+00 21+00

APPROXIMATE EXTENT OF IRRIGATED AREA (TYP)

STA 10+50
MATCHLINE
SEE DWG C-4

STA 21+50
MATCHLINE
SEE DWG C-7

PIPE

SCALE IN FEET
1" = 40 Ft.

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SHEET 7 OF XX

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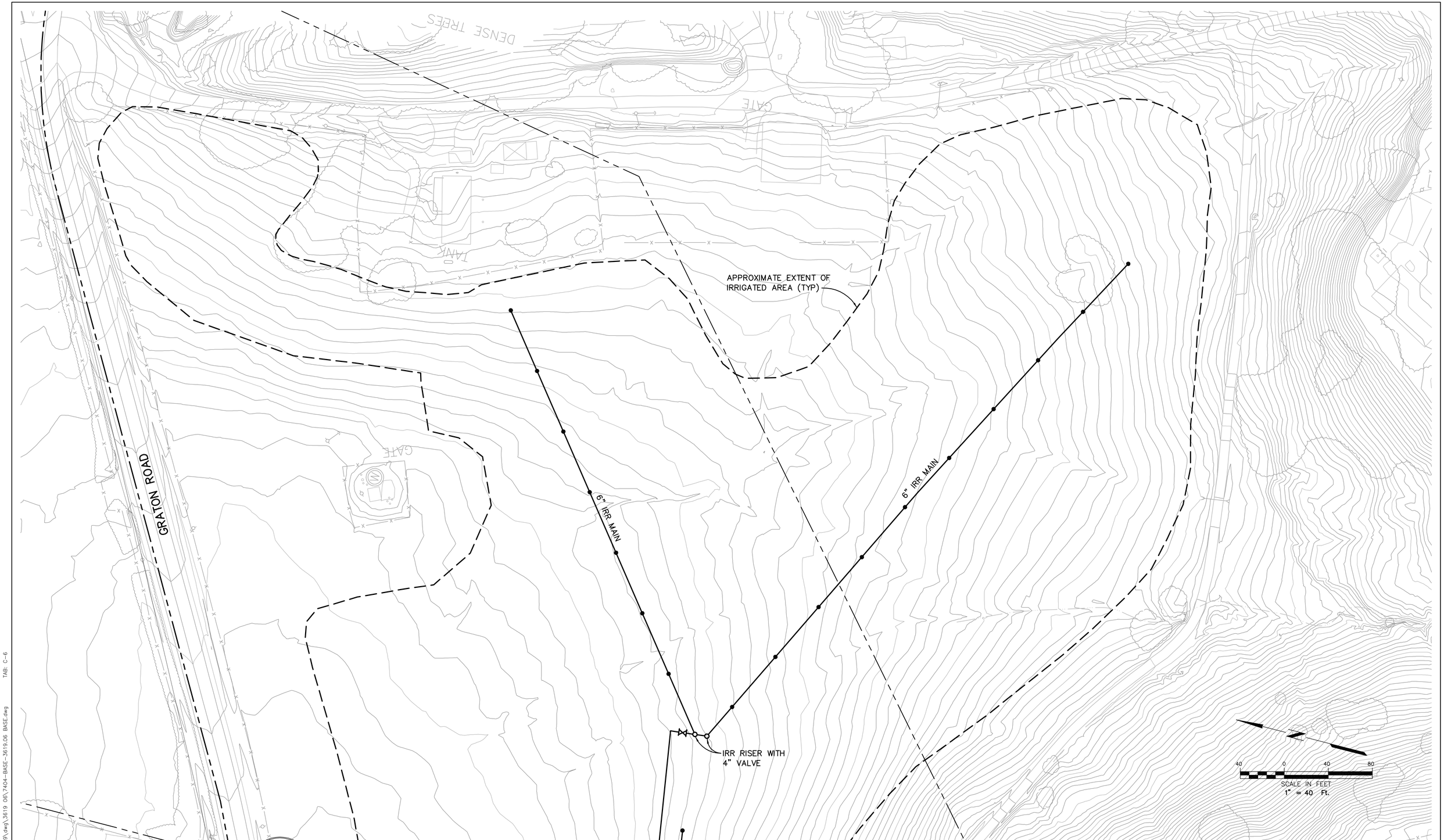


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TRANSFER MAIN & IRRIGATION
PLAN

FILE NAME: 3619.06 BASE.dwg
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SHEET 8 OF XX

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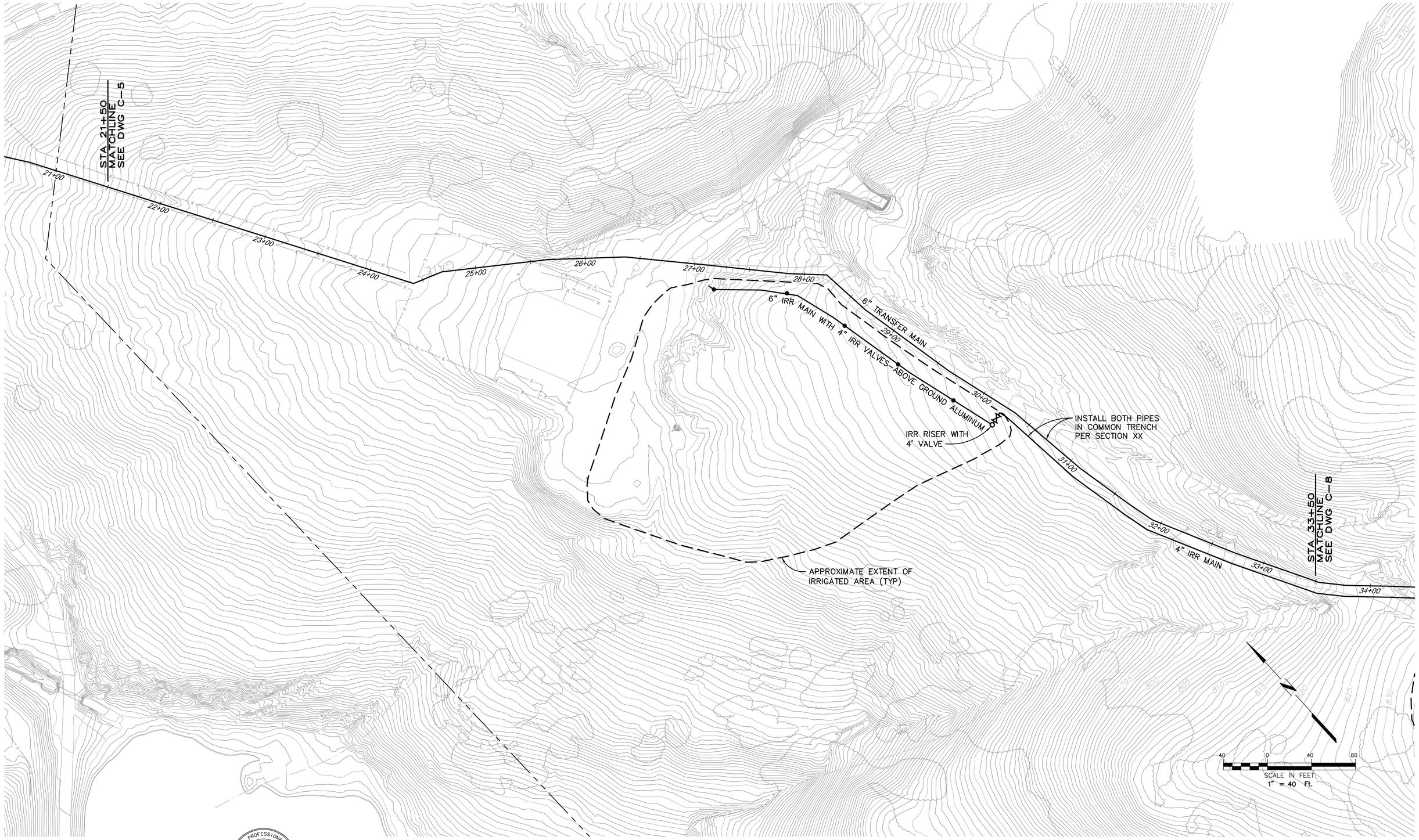


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TRANSFER MAIN & IRRIGATION
PLAN

FILE NAME: 3619.06 BASE.dwg
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TRANSFER MAIN & IRRIGATION PLAN

FILE NAME: 3619.06 BASE.dwg
CONTRACT NUMBER: 7404
DRAWING NUMBER: C-8
SHEET 10 OF XX

TAB: C-8
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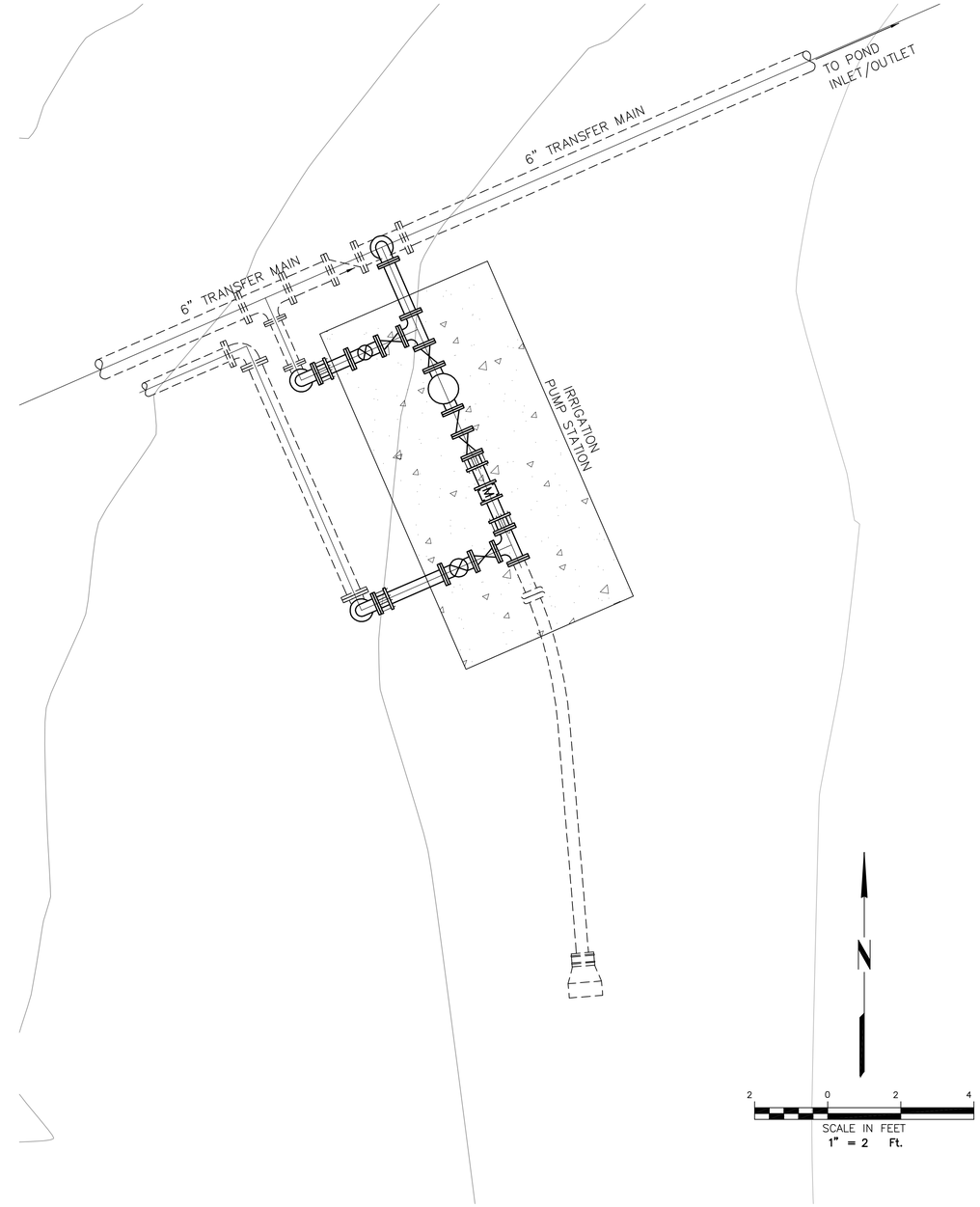


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IRRIGATION BOOSTER PUMP
STATION PLAN, SECTIONS & DETAILS

FILE NAME: 3619.06 BASE.dwg
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SHEET 11 OF XX

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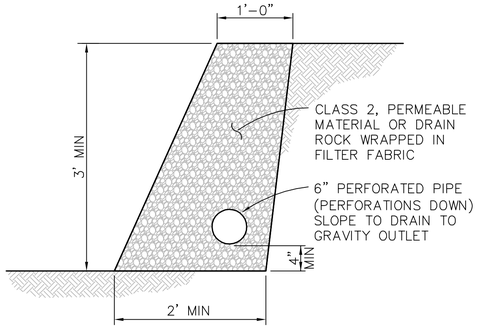
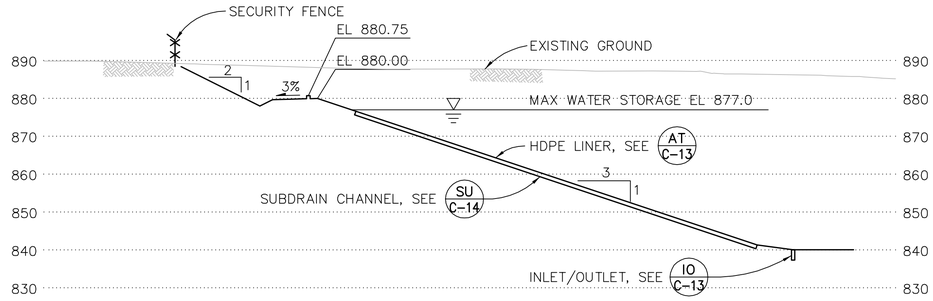
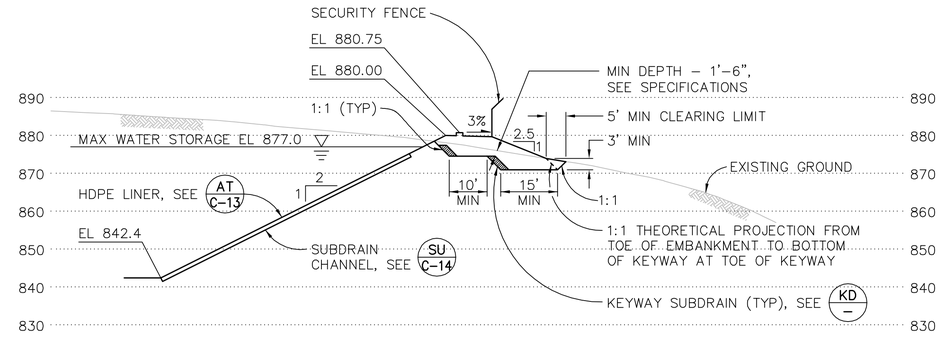


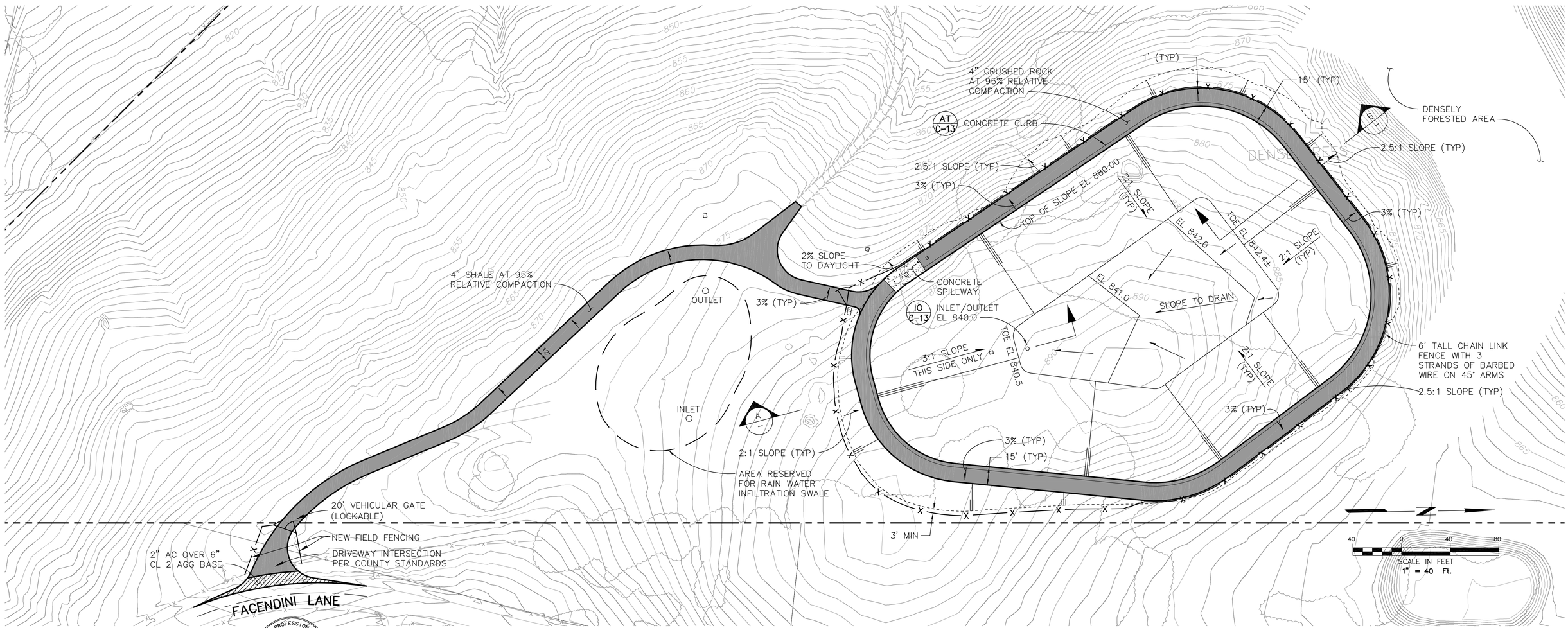

KEYWAY SUBDRAIN (KD —)
NOT TO SCALE




SECTION (A —)
SCALE: HORZ. 1"=20'
VERT. 1"=20'




SECTION (B —)
SCALE: HORZ. 1"=20'
VERT. 1"=20'

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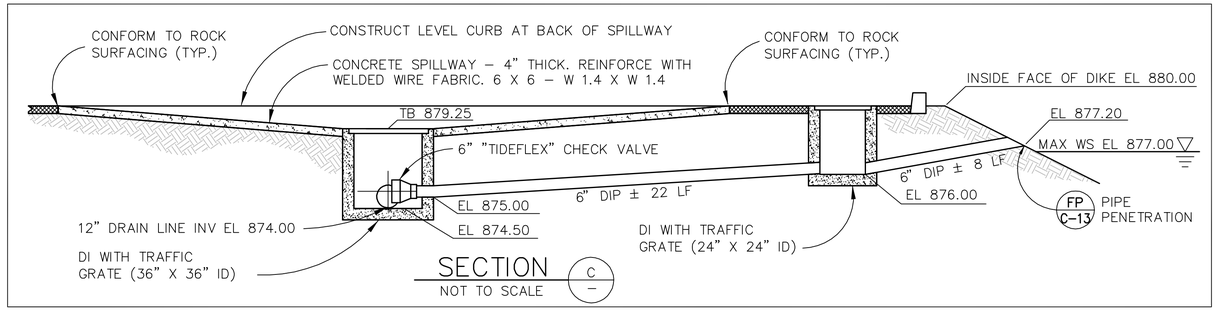
OCCIDENTAL COUNTY SANITATION DISTRICT
EFFLUENT STORAGE POND
GRADING PLAN

FILE NAME: 3619.06 BASE.dwg
CONTRACT NUMBER: 7404
DRAWING NUMBER: C-10
SHEET 12 OF XX

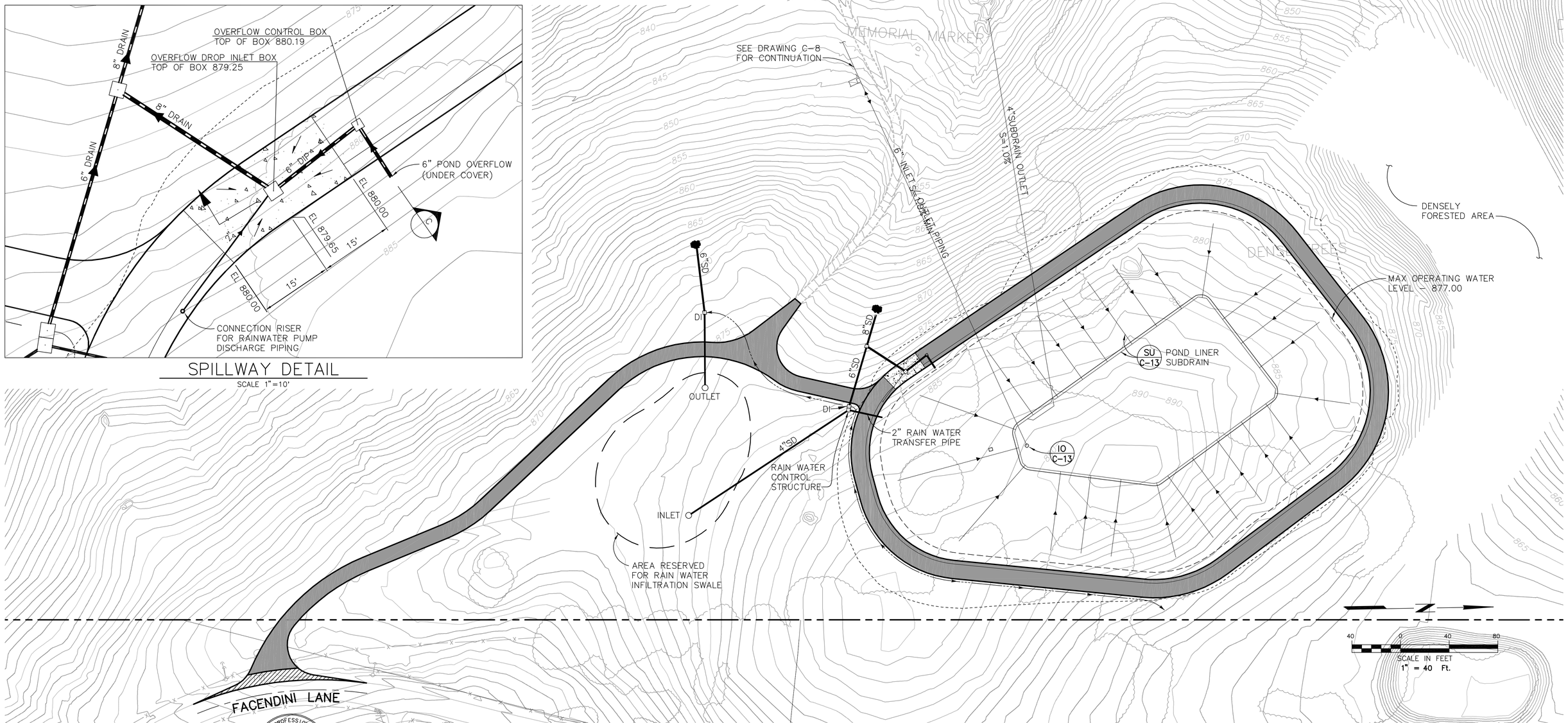
02-14-13 kilizaghi \3619\dwg\3619 06\7404-BASE-3619.06 BASE.dwg TAB: C-10







SPILLWAY DETAIL
SCALE 1"=10'




BAR LENGTH ON ORIGINAL DRAWING EQUALS ONE INCH. ADJUST SCALE ACCORDINGLY

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REGISTERED PROFESSIONAL ENGINEER RICHARD J. TINGLEY No. C 31798 Exp. 12-31-14 CIVIL STATE OF CALIFORNIA

30% SUBMITTAL

| NO. | DATE | REVISION | BY |
|---|---|---|---|
| | | | |

PRELIMINARY
SUBJECT TO REVISION

SONOMA COUNTY WATER AGENCY

SCALE: AS SHOWN
DATE: FEBRUARY 2013
DRAWN: PIT
REVIEWED:

OCCIDENTAL COUNTY SANITATION DISTRICT
EFFLUENT STORAGE POND
PIPING PLAN

FILE NAME: 3619.06 BASE.dwg
CONTRACT NUMBER: 7404
DRAWING NUMBER: C-11
SHEET 13 OF XX

TAB: C-11
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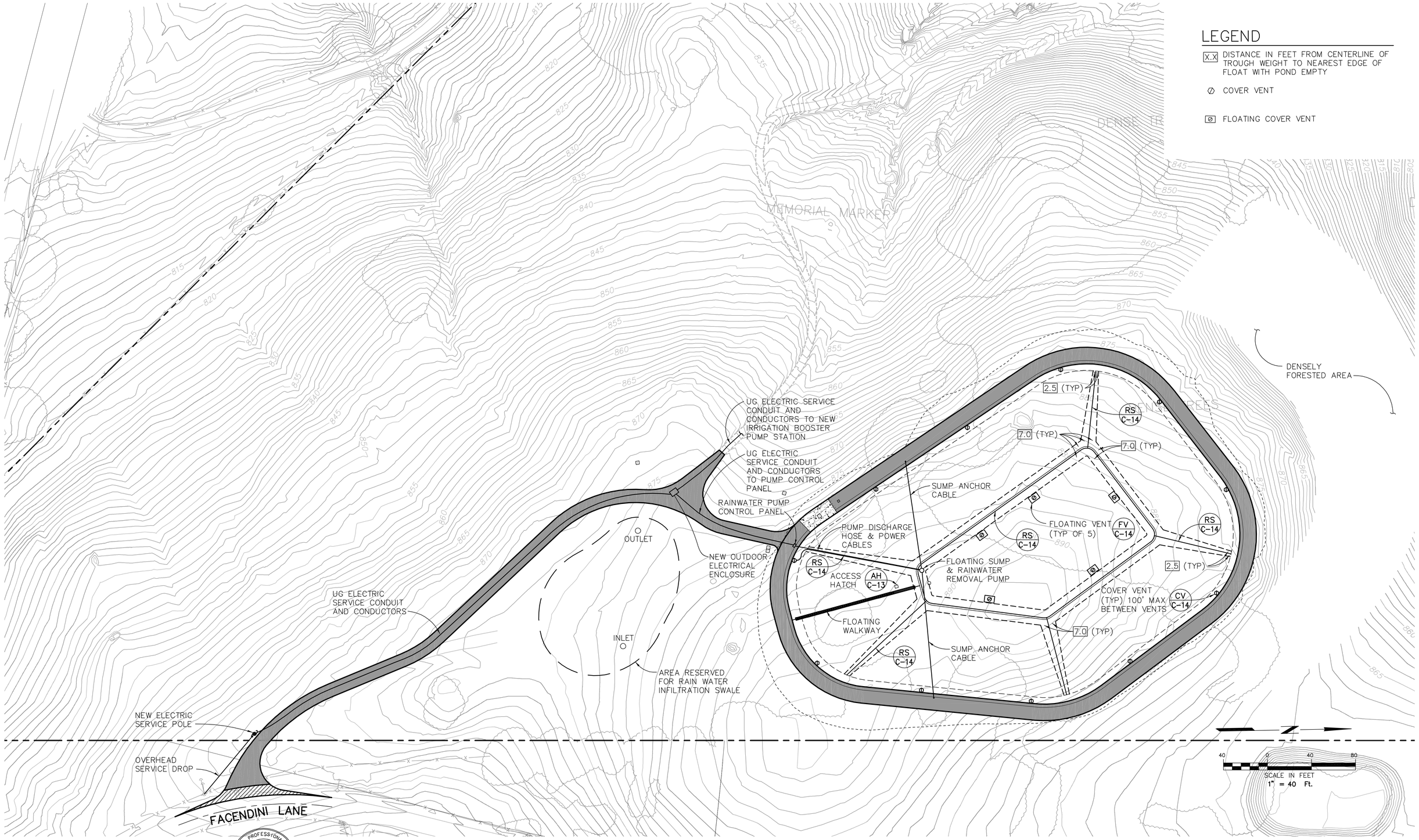
02-14-13

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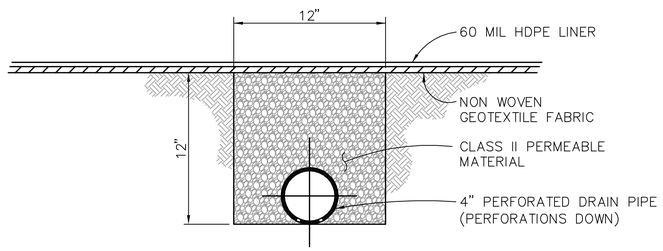
SCALE: AS SHOWN
DATE: FEBRUARY 2013
DRAWN: PIT
REVIEWED:

OCCIDENTAL COUNTY SANITATION DISTRICT
EFFLUENT STORAGE POND
COVER & ELECTRICAL PLAN

FILE NAME: 3619.06 BASE.dwg
CONTRACT NUMBER: 7404
DRAWING NUMBER: C-12
SHEET 14 OF XX

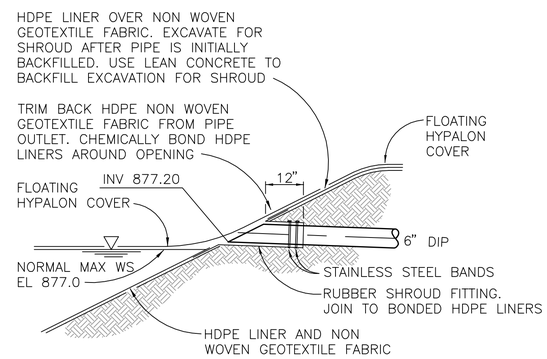
02-14-13 kf:\tgh\ \3619\dwg\3619 06\7404-BASE-3619.06 BASE.dwg TAB: C-12




## POND LINER SUBDRAIN

NOT TO SCALE

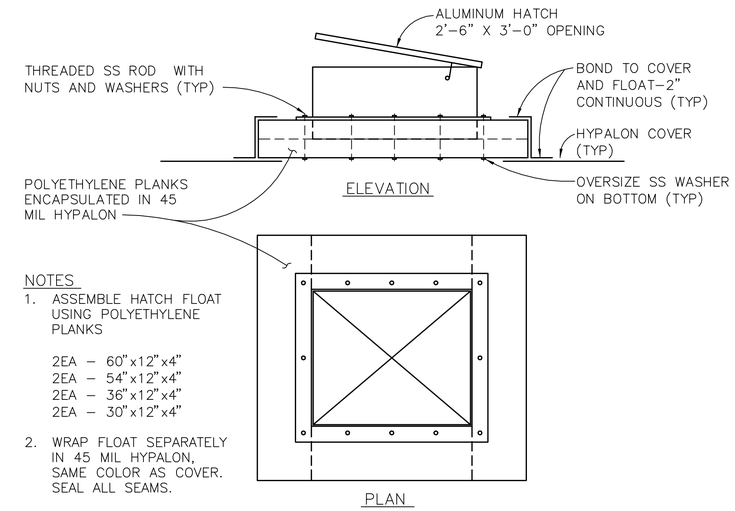
SU VAR




## FLUSH CUT PIPE PENETRATION

SCALE : 1/2" = 1'

FP C-11




NOTES

1. ASSEMBLE HATCH FLOAT USING POLYETHYLENE PLANKS

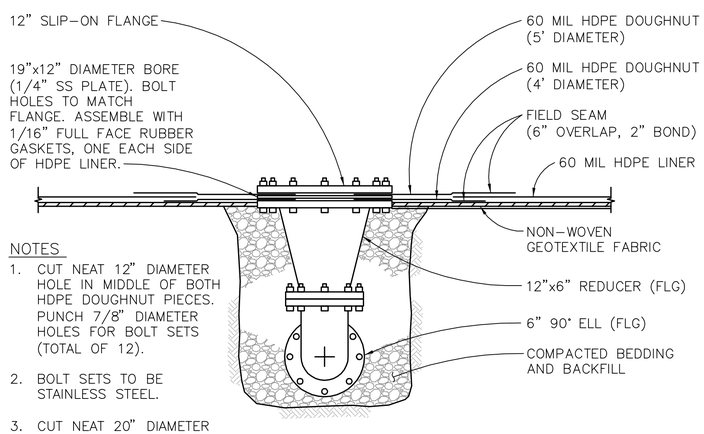
   2EA – 60"x12"x4"
   2EA – 54"x12"x4"
   2EA – 36"x12"x4"
   2EA – 30"x12"x4"

2. WRAP FLOAT SEPARATELY IN 45 MIL HYPALON, SAME COLOR AS COVER. SEAL ALL SEAMS.

## HATCH

NOT TO SCALE

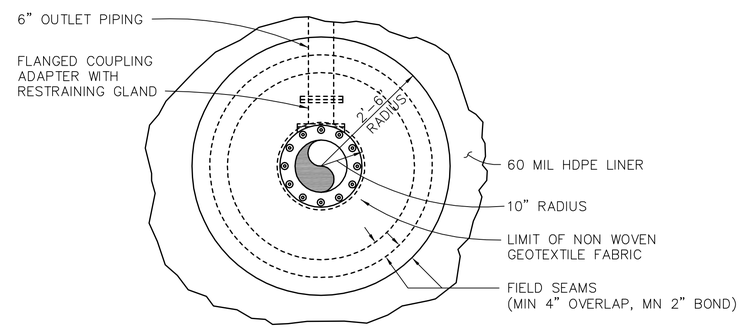
AH C-12




NOTES

1. CUT NEAT 12" DIAMETER HOLE IN MIDDLE OF BOTH HDPE DOUGHNUT PIECES. PUNCH 7/8" DIAMETER HOLES FOR BOLT SETS (TOTAL OF 12).
2. BOLT SETS TO BE STAINLESS STEEL.
3. CUT NEAT 20" DIAMETER HOLE IN NON WOVEN GEOTEXTILE FABRIC CENTERED ON FITTING.

ELEVATION

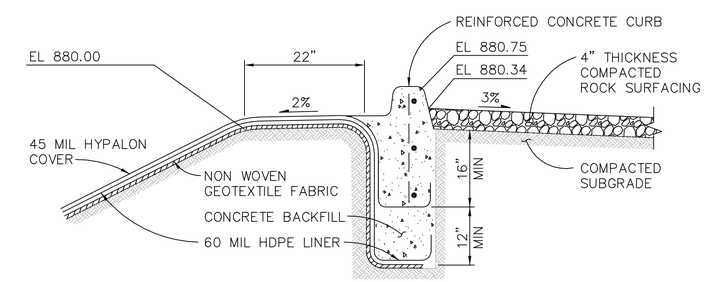



PLAN

## INLET/OUTLET

NOT TO SCALE

IO VAR




## POND LINER ANCHOR TRENCH

NOT TO SCALE

AT C-10

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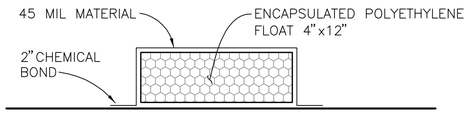
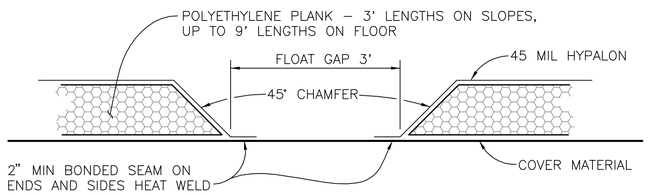
OCCIDENTAL COUNTY SANITATION DISTRICT
EFFLUENT STORAGE POND
DETAILS

FILE NAME: 3619.06 DETAIL.dwg
CONTRACT NUMBER: 7404
DRAWING NUMBER: C-13
SHEET 15 OF XX

02-14-13 kfilippi \3619\dwg\3619.06\7404-DETAIL-3619.06 DETAIL.dwg TAB: C-13

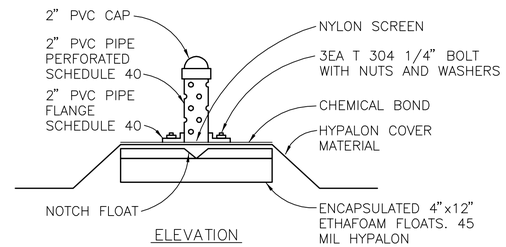



## TYPICAL FLOAT SECTION
NOT TO SCALE



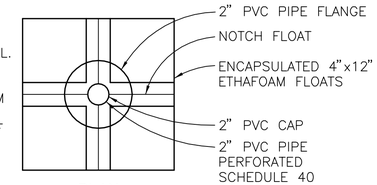

NOTE:
FLOAT GAP IS 3' BUT CAN VARY SOMEWHAT DUE TO FIELD CONDITIONS.

## TOP SIDE FLOATS
NOT TO SCALE



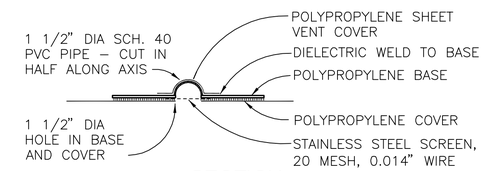

ELEVATION

NOTES
1. MOUNT FLANGE TO A PIECE OF COVER MATERIAL.
2. MAKE A CUT IN COVER. SLIDE FLOAT UNDER SEAM COVER TO FLAT. SEAM FLANGE SKIRT TO TOP OF COVER.

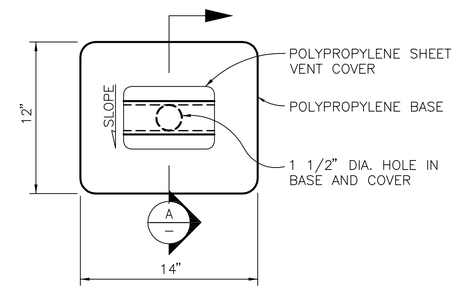



PLAN

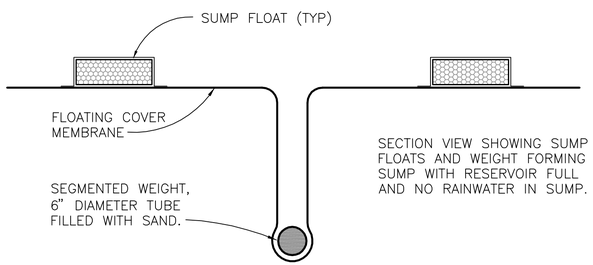
## FLOATING VENT
NOT TO SCALE (FV / C-12)



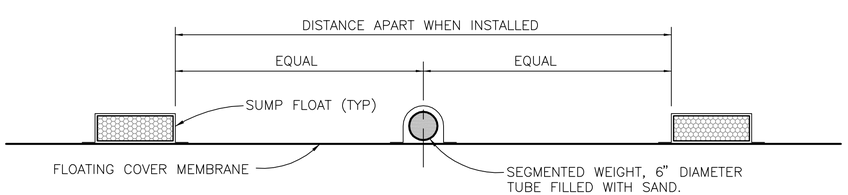

## SECTION
NOT TO SCALE (A / –)




## COVER VENT
NOT TO SCALE (CV / C-12)

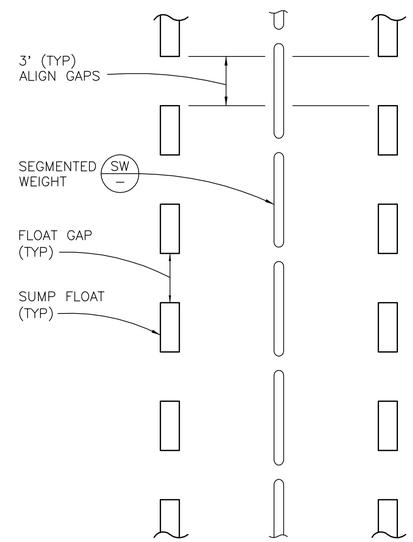



## RAINWATER SUMP SECTION
NOT TO SCALE (RS / C-12)

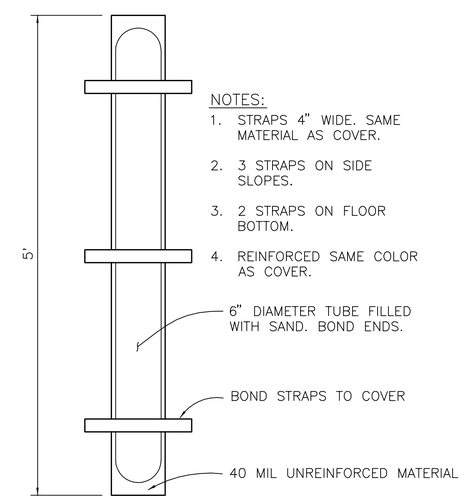



SECTION VIEW SHOWING INSTALLATION OF SUMP WITH COVER FLAT ON POND BOTTOM AND SIDES

## RAINWATER SUMP SECTION
NOT TO SCALE (RS / C-12)




## FLOAT GAPS / SEGMENTED WEIGHT
NOT TO SCALE




NOTES:
1. STRAPS 4" WIDE. SAME MATERIAL AS COVER.
2. 3 STRAPS ON SIDE SLOPES.
3. 2 STRAPS ON FLOOR BOTTOM.
4. REINFORCED SAME COLOR AS COVER.

## SEGMENTED WEIGHT
NOT TO SCALE (SW / –)

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PRELIMINARY
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SONOMA COUNTY WATER AGENCY

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EFFLUENT STORAGE POND
COVER DETAILS

FILE NAME: 3619.06 DETAIL.dwg
CONTRACT NUMBER: 7404
DRAWING NUMBER: C-14
SHEET 16 OF XX

02-14-13 kfilizgni \3619\dwg\3619 06\7404-DETAIL-3619.06 DETAIL.dwg TAB: C-14

# APPENDIX G

# ENGINEERING CALCULATIONS – PUMPING AND IRRIGATION

**Transfer from WWTP to Pond C**

**Given:**

| | |
|---|---|
| 6 | inch PVC & DIP main |
| 140 | gpm capacity of existing disinfection pumps (DM, per Hody Wilson, email 3/5/12) |
| 794 | feet ground elevation at CCTs (SCWA survey) |
| 877 | max WSE at pond C North |
| 4800 | feet pipe from WWTP to Pond C North |

**Assume**

No increase in disinfection flow

| | |
|---|---|
| 1.1 | multiplier on transfer pump rate to allow for variation in disinfection flows |
| 1 | foot minimum water depth in pump control tank |

**Then:**

| | |
|---|---|
| 795 | Feet minimum operating WSE in control tank |

**Use Hazen-Williams:** hl=10.44*L*(q/C)^1.85/d^4.8655

| | | |
|---|---|---|
| Q = | 154 | gpm |
| Set C = | 130 | |
| vel = | 1.75 | fps |
| Entr Hl = | 0.05 | feet |

**Pump to Pond C:**

| | | |
|---|---|---|
| Hf = | 12 | feet |
| delta Z = | 82 | feet |
| TDH = | 94 | feet |

| | |
|---|---|
| 0.75 | assumed pump efficiency |
| 0.9 | assumed motor efficiency |
| **5.4** | **HP pump needed** **SAY** **7.5 HP** |

**Size control tank:**

| | |
|---|---|
| 140 | gpm into tank |
| 154 | gpm pumped from tank |
| 4 | = max no. pump starts/hr |
| | |
| 14 | gpm greater outflow than inflow |
| 210 | gallons minimum between pump on and pump off |

Use 500 gallon tank to allow minimum and max fill, reduce pump cycling.

**Irrigation Areas and Flow Rates**

| | |
|---|---|
| Peak Monthly Irrigation Rate | 4,400,000 gal |
| Total Irrigation Area | 24 |

| | Pond Irrigation Area | Central Irrigation Area | Large Irrigation Area | South Irrigation Area |
|---|---|---|---|---|
| Irrigation Service Area | 200,000 sq ft | 65,900 sq ft | 659,000 sq ft | 122,000 sq ft |
| | 4.59 acres | 1.51 acres | 15.13 acres | 2.80 acres |
| Monthly Irrigation Volume | 840,577 gallons | 276,970 gallons | 2,769,701 gallons | 512,752 gallons |
| Irrigating hours | 5.75 hr/d | 7.5 hr/d | 6.75 hr/d | 7.25 hr/d |
| Throw Radius of Sprinkler | 50 feet | 50 feet | 50 feet | 50 feet |
| Total number of heads | 60 | 16 | 189 | 27 |
| Number of Zones | 4 | 1 | 6 | 1 |
| Area per zone | 50,000 sq ft | 65,900 sq ft | 109,833 sq ft | 122,000 sq ft |
| Heads per zone | 15 | 16 | 32 | 27 |
| Irrigation Volume per Irrigation | 48,871 gallons | 64,412 gallons | 107,353 gallons | 119,245 gallons |
| Flow Rate | 142 gpm | 143 gpm | 265 gpm | 274 gpm |
| Daily Irrigation Rate | 1.6 in/d | 1.6 in/d | 1.6 in/d | 1.6 in/d |
| Hourly Irrigation Rate | 0.27 in/hr | 0.21 in/hr | 0.23 in/hr | 0.22 in/hr |
| Volume per head | 3,258 gallons | 4,026 gallons | 3,408 gallons | 4,416 gallons |
| Flow rate per head | 9.4 gpm | 8.9 gpm | 8.4 gpm | 10.2 gpm |

| Pumped Area/High Zone | | |
|---|---|---|
| Pond Irrigation Area | | |
| elevation at pump | 845 ft | |
| max irrigation elevation | 890 ft | |
| min irrigation elevation | 830 ft | |
| Distance to max | 650 ft | |
| Distance to min | 200 ft | |
| Pipe Diameter | 4 in | |
| Head loss to max | 7.3 ft | |
| Head loss to min | 2.3 ft | |
| Sprinkler Pressure at max | 40 psi | w/spreader 3/16" |
| Head needed | 144 ft | |
| Input to Motor | 7.2 Hp | |
| Available Pressure at min | 68 psi | w/plug 13/64" reduce to 55 psi |
| | | |
| Central Irrigation Area | | |
| max irrigation elevation | 800 ft | |
| min irrigation elevation | 760 ft | |
| Distance to max | 600 ft | |
| Distance to min from max | 400 ft | |
| Pipe Diameter | 4 in | |
| Head loss to max | 6.8 ft | |
| Head loss to min | 4.5 ft | |
| Available Pressure at max | 65 psi | w/plug 3/16" |
| Available Pressure at min | 81 psi | w/plug 3/16" |

| | | Gravity Area | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Distance | Flow Rate | Pipe Diameter | Loss | Max head | min head | Elevation | Available Pressure | | Nozzle Type |
| | | | | | | | | | | Max | Min | |
| | | Pond Water Surface | | | | | 877 ft | 840 ft | | | | |
| | 1 | Tee to pump | 325 ft | 417 gpm | 6 in | 4 ft | 873 ft | 836 ft | 830 | 19 psi | 3 psi | reduce to 3 psi |
| | 2 | Tee to Irrigation | 1700 ft | 274 gpm | 6 in | 9 ft | 828 ft | 827 ft | 665 ft | 71 psi | 71 psi | w/ plug 3/16" |
| | 3 | Tee to high area | 430 ft | 274 gpm | 6 in | 2 ft | 826 ft | 825 ft | 660 ft | 72 psi | 72 psi | w/ plug 3/16" |
| 8.4 gpm | 4 | Branch to High Area | 700 ft | 274 gpm | 6 in | 4 ft | 822 ft | 822 ft | 685 ft | 60 psi | 59 psi | w/ plug 3/16" |
| | 5 | High Area | 100 ft | 46 gpm | 3 in | 1 ft | 822 ft | 821 ft | 690 ft | 57 psi | 57 psi | w/ plug 3/16" |
| | 6 | Branch to Low Area | 600 ft | 274 gpm | 6 in | 3 ft | 825 ft | 824 ft | 660 ft | 72 psi | 71 psi | w/ plug 3/16" |
| | 7 | Low Area | 300 ft | 46 gpm | 3 in | 2 ft | 823 ft | 823 ft | 640 ft | 80 psi | 79 psi | w/ plug 3/16" |
| | 8 | Tee to South Area/high south area | 250 ft | 274 gpm | 6 in | 1 ft | 822 ft | 821 ft | 655 ft | 73 psi | 72 psi | w/ plug 13/64" |
| 10.2 gpm | 9 | Branch to Low South Area | 570 ft | 274 gpm | 4 in | 21 ft | 801 ft | 800 ft | 630 ft | 74 psi | 74 psi | w/ plug 13/64" |
| | 10 | Low South Area | 60 ft | 274 gpm | 3 in | 9 ft | 791 ft | 791 ft | 630 ft | 70 psi | 70 psi | w/ plug 13/64" |