PREPARED FOR
CITY OF MORRO BAY
AND
CAYUCOS SANITARY DISTRICT

Wastewater Treatment Plant Upgrade Project

JULY 2010

Facility Master Plan
Draft Amendment No. 2

MWH®

BUILDING A BETTER WORLD

in association with

EDA, Inc.
Fugro West, Inc.
RRM Design Group

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Executive Summary

The City of Morro Bay and the Cayucos Sanitary District (MBCSD) have embarked on a program to upgrade their existing wastewater treatment plant (WWTP) to comply with full secondary treatment standards. Critical Project milestones are outlined in the Conversion Schedule contained in the NPDES Permit issued by the Regional Water Quality Control Board (RWQCB). The Conversion Schedule set forth in the Settlement Agreement with the RWQCB establishes that the MBCSD WWTP will be upgraded to provide full secondary treatment by March 31, 2014.

The WWTP Upgrade Project will be advanced in three phases:

- Planning and environmental evaluation phase with preparation of Facility Master Plan (FMP) and Environmental Impact Report (EIR), respectively.
- Final design phase with preparation of Contract Documents (specifications and drawings).
- Bidding and construction phase with demolition of the existing plant facilities after startup of the new facilities.

The purpose of this draft document, "Facility Master Plan - Amendment No. 2", is to finalize the FMP and to provide a project description for the EIR to facilitate completion of the planning and environmental phase of the Project. The general design criteria and treatment technology presented in Amendment No. 2 is similar to that presented in the draft FMP – Amendment No. 1. Design flows and loadings have been updated to reflect longer periods of record. The conceptual plant configuration has been adjusted to consolidate facilities and improve plant hydraulics. This amendment supplements, and where appropriate, supersedes details of the draft FMP and the draft FMP – Amendment No. 1 that were previously prepared.

The recommended WWTP Upgrade Project is based on the utilization of oxidation ditch treatment technology with tertiary filtration and project delivery with a conventional design-bid-build (DBB) approach. The proposed Project will consist of a new WWTP to replace the existing WWTP which must be demolished to mitigate flooding. The new WWTP will be constructed immediately to the south of the existing plant in the vicinity of the existing sludge drying beds and will be placed on engineered fill above the 100-year flood elevation.

The estimated project cost of the WWTP Upgrade Project recommended in the draft FMP – Amendment No. 1 was approximately $27,400,000 with a corresponding estimated construction cost of $20,300,000 in June 2006 dollars (ENR Index of 7700). When escalated to June 2010 dollars (ENR Index of 8800), the estimated project cost of the WWTP Upgrade Project is approximately $31,300,000 with a corresponding estimated construction cost of $23,200,000. These costs will be updated upon completion of the first phase of design (Preliminary Design).

Upon acceptance of the FMP – Amendment No. 2 and the draft EIR by MBCSD, the final design phase of the WWTP Upgrade Project can be authorized. The Project is currently on track to meet the tight compliance schedule.

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FMP - Amendment No. 2

Introduction

The City of Morro Bay and Cayucos Sanitary District (MBCSD) have commenced the planning and design of the Wastewater Treatment Plant (WWTP) Upgrade Project with the development of the following documents:

- Draft Facility Master Plan (FMP) entitled "Wastewater Treatment Plant Facility Master Plan Report" dated September 2007 prepared by Carollo Engineers.
- Draft FMP – Amendment No. 1 entitled "Facility Master Plan Amendment No. 1" dated August 2009 prepared by Carollo Engineers.
- Draft Environmental Impact Report for the subject Project is currently under preparation by ESA.

The design team of MWH with EDA (surveying), Fugro (geotechnical), and RRM (landscape architecture and architecture) has been contracted by MBCSD to prepare a FMP – Amendment No. 2 as presented herein to complete the Facility Master Plan. The completed Facility Master Plan will consist of the draft Facility Master Plan, draft FMP – Amendment No. 1, and FMP – Amendment No. 2. Note that where differences occur between the above documents, the more recent document will prevail. The completed FMP will be coordinated with completion of the EIR. The completed FMP and EIR will be issued for public review and permitting processes after acceptance by MBCSD.

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Work to Date

Draft FMP. The draft FMP was predicated on an upgrade and rehabilitation of the existing WWTP. The existing WWTP consists of the following facilities:

- Headworks with influent screening, pumping, and aerated grit basin
- Primary clarifiers
- Trickling filters
- Solids contact aeration basin
- Secondary clarifier
- Chlorine contact basin
- Anaerobic digesters
- Sludge drying beds.

The draft FMP WWTP recommended retirement of the following facilities:

- Primary clarifiers
- Trickling filters
- Solids contact aeration basin
- Chlorine contact basin
- Digester No. 1.

The draft FMP recommended construction of the following facilities:

- Rehabilitation of existing headworks
- New oxidation ditches
- New secondary clarifier to operate in conjunction with the existing secondary clarifier
- New tertiary filtration
- New chlorine contact basin
- Reuse of existing chlorine building
- New sludge thickening
- Rehabilitation of Digesters No. 2 and No. 3
- New sludge dewatering
- Use of existing sludge drying beds for partial composting.

Figure 1 shows a conceptual site plan of the recommended project that was presented in the draft FMP.

Draft FMP – Amendment No. 1. During the preparation of the draft EIR, a flood hazard analysis of the existing WWTP was conducted. A report entitled "Wastewater Treatment Plant Flood Hazard Analysis" dated August 7, 2009 prepared by the Wallace Group concluded that the existing WWTP site was subject to inundation from the 100-year storm event to depths ranging from 3 to 4.5 feet. After review of various alternatives to mitigate the potential flooding, relocation of the WWTP facilities was recommended. The new location is immediately to the south of the existing facilities and would be developed with engineered fill to raise the finished grade above the 100-year flood elevation.

The relocation of the WWTP facilities would require the demolition of the existing WWTP to preserve a floodway that would not worsen the flood elevations experienced by adjoining properties during storm events. The demolition of existing facilities would preclude their rehabilitation and continued use. Consequently, the draft FMP – Amendment No. 1 was prepared to evaluate alternatives for a revised approach to upgrade the existing WWTP.

The draft FMP – Amendment No. 1 recommended demolition of the following facilities:

- Headworks
- Primary clarifiers
- Trickling filters
- Solids contact aeration basin
- Secondary clarifier
- Chlorine contact basin
- Chlorination building
- Air release structure
- Anaerobic digesters
- Sludge drying beds
- Administration Building
- Maintenance Building

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The draft FMP – Amendment No. 1 recommended construction of the following facilities:

- New influent pump station
- New screening facility
- New oxidation ditches
- New secondary clarifiers
- New RAS/WAS pump station
- New tertiary filtration
- New chlorine contact basin
- New chlorination building
- New air release structure
- New sludge thickening
- New thickened sludge holding tank
- New sludge dewatering
- New electrical building
- New standby power facility
- New Administration Building
- New Maintenance Building
- Relocation of household hazardous waste station

Figure 2 shows a conceptual site plan of the recommended project that was presented in the draft FMP – Amendment No. 1.

The project cost of the WWTP upgrade recommended in the draft FMP was estimated to be approximately $28,000,000 of which approximately $4,000,000 was allocated as a contingency for flood mitigation. The project cost of the WWTP upgrade recommended in the draft FMP – Amendment No. 1 was estimated to be approximately $27,400,000.

The incremental costs of additional facilities needed to replace facilities that would be demolished under draft FMP – Amendment No. 1 were largely offset by a reduction in the solids handling facilities. Anaerobic digestion and partial composting would be discontinued and the transportation of unclassified sludge for third party processing and disposal would be implemented.

FMP – Amendment No. 2. The intent of FMP - Amendment No. 2 is to achieve the following:

- Finalize the FMP as an aggregate document consisting of the draft FMP, draft FMP – Amendment No. 1, and FMP – Amendment No. 2.
- Provide project description for the draft EIR.
- Revise the flow and loading parameters by evaluation of longer periods of record for historical data.
- Incorporate adjustments to the treatment processes and conceptual site layout identified during the interview and negotiation process with the design consultant (MWH) and during the development of Amendment No. 2.

The revisions to the flow and loading parameters are discussed under the Project Description presented herein.

The key adjustments to the conceptual design presented in FMP – Amendment No. 1 are summarized as follows:

- Add grit removal as a pretreatment component to minimize deposition of grit in downstream facilities (oxidation ditches) that would take up valuable hydraulic basin volume. Periodic removal of the grit from the Oxidation Ditches would be disruptive to on-going plant operations and would be difficult to remove.
- Consolidate screenings, grit removal, and sludge dewatering in a common Residuals Facility to centralize truck traffic and better accommodate the potential for future odor control if desired in the future.
- Design Residuals Facility so that the screenings, grit, and sludge processes can be enclosed in a building with future construction if needed.

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- Add a Secondary Pump Station to lift secondary effluent from the Secondary Clarifiers to the Tertiary Filter and Chlorine Contact Basin to reduce the height of the upstream facilities (Residuals Facility, Oxidation Ditches, and Secondary Clarifiers) and accommodate the hydraulic profile of the existing outfall.
- Provide a two-chamber chlorine contact basin to facilitate operation and maintenance.
- Design Tertiary Filter and Chlorine Contact Basin to accommodate the future implementation of Title 22 Reclaimed Water capability.
- Delete sludge thickening and thickened sludge storage facilities. Sequential thickening and dewatering can be difficult because the sludge conditioning for thickening can interfere with the sludge conditioning for dewatering. The storage of thickened sludge is problematic because mixing and aeration difficulties can lead to significant odor potential. The dewatering of waste activated sludge (WAS) directly, without thickening, has been successfully accomplished at many existing oxidation ditch plants.
- Add provisions for temporary sludge dewatering at the existing WWTP during construction of the WWTP Upgrade. The new WWTP will be constructed over the existing sludge drying beds and the drying beds will not be available for use during construction.
- Locate Maintenance Building in close proximity to Residuals Facility and Chemical Station to consolidate vehicle traffic associated with hauling residuals (e.g., screenings, grit, and sludge), chemical deliveries (e.g., polymer, hypochlorite, and bisulfite), and service deliveries (e.g., spare parts and lubricants). The Maintenance Building will also house the electrical room and the proposed proximity to nearby treatment facilities will more efficiently serve the associated major electrical loads.
- Locate Operations Building and Household Hazardous Waste station for accessibility and visibility for the public and visitors. Provide public access road for the public that is separate from service access road for staff and truck traffic associated with the treatment facilities.

A conceptual site plan and flow schematic of the project recommended in the FMP – Amendment No. 2 is shown in **Figure 3** and **Figure 4**, respectively. Note that the size, location, and orientation of the facilities shown in the conceptual site plan are subject to further refinement during the final design phase of the Project.

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Flow and Loadings

Revised Flow and Loadings. The influent wastewater flow and loadings presented in the draft FMP were established for a combined MBCSD build-out population that would be reached by Y2021 for the City of Morro Bay and Y2015 for the Cayucos Sanitary District. The flow and loading projections were based on historical record information gathered for a 5-year period from Y2002 through Y2006 at the time the draft FMP was prepared.

MBCSD staff expressed concern that this period of record is too limited for the following reasons:

- Annual flow and loading data are now available for Y2007 to Y2009.
- The period of record for Y2002 – Y2006 were relatively dry years.

Consequently, flow data from Y1995 through Y2009 (15 years) were gathered and evaluated to establish the basis of design for flow parameters. Loading data for BOD and TSS for the period from Y2002 through Y2009 (8 years) were also gathered and evaluated. The revised flow and loading parameters that were developed from the re-evaluation are summarized in Table A. The corresponding flow and loading parameters presented in the draft FMP are also shown for comparison.

Table A – Revised Flow and Loadings

Parameter	Unit	Draft FMP (a)	Amend. No. 2
Period of Record			
Flow	---	Y2002 – Y2006	Y1995 – Y2009
BOD and TSS	---	Y2002 – Y2006	Y2002 – Y2009
Flow			
Annual Average Day (AADF)	mgd	1.33	1.5
Avg. Peak Season Dry Weather (PSDWF)	mgd	1.47	1.5
Average Day Maximum Month (ADMMF)	mgd	1.62	2.9
Peak Hour (PHF)	mgd	5.97	8.0
Biochemical Oxygen Demand (BOD)			
Annual Average Day	lb/day	3,800	3,500
Annual Max 30-Day	lb/day	---	4,700
Peak Season Dry Weather, Average Day	lb/day	5,000	4,200
Peak Season Dry Weather, Max Month	lb/day	----	5,500
Total Suspended Solids (TSS)			
Annual Average Day	lb/day	4,000	3,800
Annual Max 30-Day	lb/day	---	5,300
Peak Season Dry Weather, Average Day	lb/day	5,200	4,500
Peak Season Dry Weather, Max Month	lb/day	---	6,300
Total Kjeldahl Nitrogen (TKN) (b)			
Annual Average Day	lb/day	---	600
Annual Max 30-Day	lb/day	---	800
Peak Season Dry Weather, Average Day	lb/day	---	710
Peak Season Dry Weather, Max Month	lb/day	---	940

(a) From draft FMP Tables 3.11, 3.12, and 3.20.

(b) Note that TKN values are currently derived by assuming a TKN / BOD ratio of 0.17. Supplemental testing of influent wastewater samples will be conducted during Y2010 to obtain TKN data and modify the listed values if needed.

As can be seen, the inclusion of a longer period of record that captured prior wet weather years with higher rainfall has increased the values of flow parameters. The impact is small for average flow values, but more dramatic for peak flow values. The impact on BOD and TSS loadings by capturing more recent years does not have a major impact. The latter is expected given that the resident population has been relatively constant. Further information regarding the approach used for the evaluation of existing flow and loadings and the determination of the design flow and loadings is presented in a technical memorandum to MBCSD entitled "Revised Flow and Loadings" dated June 21, 2010 and prepared by MWH that is attached as Appendix A.

Project Description

Treatment Requirements and Objectives. The current WWTP is required to meet the current NPDES Permit discharge requirements presented in **Table B**. Note that the requirements listed in **Table B** do not include the discharge limitations for such constituents as metals, non-carcinogens, carcinogens, and others for brevity and because the history of compliance with these constituents has not been an issue. The current 301(h) modified NPDES Permit addresses BOD and TSS limitations only. The existing WWTP was designed to meet all secondary treatment and the California Ocean Plan requirements.

Table B – NPDES Discharge Requirements

Parameter	Unit	Value
Monthly Dry Weather Flow	**mgd**	**2.36**
Biochemical Oxygen Demand (BOD)		
Average Monthly	mg/L	120
Instantaneous Maximum	mg/L	180
30-Day Average Percent Removal	%	30
Total Suspended Solids (TSS)		
Average Monthly	mg/L	70
Instantaneous Maximum	mg/L	105
30-Day Average Percent Removal	%	75
Total Coliform Bacteria		
30-Day Median	MPN / 100 ml	23
Maximum	MPN / 100 ml	2400
Grease and Oil		
Average Monthly	mg/L	25
Average Weekly	mg/L	40
Instantaneous Maximum	mg/L	75
Settleable Solids		
Average Monthly	ml/L	1.0
Average Weekly	ml/L	1.5
Instantaneous Maximum	ml/L	3.0
Turbidity		
Average Monthly	NTU	75
Average Weekly	NTU	100
Instantaneous Maximum	NTU	225
pH		**6.0 – 9.0**

The MBCSD has committed to full secondary treatment objectives that are significantly more restrictive than the NPDES Discharge Requirements listed in **Table B** for BOD and TSS. The treatment objectives of the MBCSD are to provide secondary treatment at all times and to provide tertiary filtration capacity equivalent to the peak season dry weather (PSDW) flow of 1.5 mgd. The secondary effluent quality suitable for routine tertiary filtration would dictate effluent BOD/TSS in the range of 10/10 to 15/15 mg/L. The tertiary effluent quality for BOD/TSS would be expected to be in the range of 2/2 to 5/5 mg/L.

Treatment Concept. The new WWTP will replace the existing WWTP that will be retired and demolished to mitigate area flooding during major storm events. An overview of the treatment concept for the proposed treatment processes is presented below. A more detailed discussion of each treatment process will be presented elsewhere. A general flow schematic of the recommended WWTP is presented in **Figure 4**.

Influent wastewater is delivered from the collection systems to the WWTP via gravity sewers and will be lifted with a new Influent Pump Station to the Residuals Facility. The Residuals Facility will include screening and grit removal units for pretreatment of the influent wastewater to remove inert debris, rags, plastics, and grit for protection of the downstream treatment processes. The Residuals Facility will also house the sludge dewatering facilities.

The cornerstone of the new WWTP will be the utilization of the oxidation ditch treatment technology. Oxidation ditches utilize an extended aeration activated sludge process with secondary clarifiers to achieve excellent removals of BOD, TSS, and NH3 and to produce a high quality effluent suitable for tertiary filtration. The extended aeration will be supplemented with an anaerobic zone to produce a better settling sludge for reliable secondary clarifier performance and an anoxic zone to reduce aeration demand and recover alkalinity. The oxidation ditch technology provides reliable, robust biological treatment, can readily handle peak flows and loadings, and is easy to operate and maintain.

Tertiary filtration capacity that matches the average PSDW flow capacity of the new WWTP will be provided to further improve effluent quality. The effluent will be treated with a chlorine contact basin that utilizes hypochlorite (bleach) for disinfection and bisulfite to remove any chlorine residual prior to discharge. The treated effluent will be discharged to the existing ocean outfall system.

The new WWTP will include a new Operations Building to house administrative offices, laboratory, control room, locker rooms, break room, and conference room and a new Maintenance Building to house workshop, tools, spare parts, and electrical room. A standby power facility will be provided to provide electrical power during power outages. Space and access road will be provided for the relocation of the existing Household Hazardous Waste station.

Hydraulic Profile. The hydraulic profile of the new WWTP is critical for development of the location and configuration of the treatment facilities. The influent wastewater must be lifted from the influent sewer for delivery to the series of treatment processes and ultimately to reach the existing ocean outfall. The hydraulic profile presented in **Figure 5** tabulates the preliminary water surface elevations needed to convey wastewater through the treatment processes and in turn determines the height of the associated structures and hydraulic basins.

A Secondary Pump Station is recommended to lift secondary effluent from the Secondary Clarifiers to the Tertiary Filter and Chlorine Contact Basin for subsequent delivery by gravity to the existing effluent outfall. The influent wastewater could be lifted high enough by the Influent Pump Station to flow by gravity through all the downstream treatment facilities without a Secondary Pump Station. This will be reviewed in greater detail during final design. However, based on current information, the resultant height of the structures, in particular the Residuals Facility, to accommodate a single pump station configuration would be expected to be more expensive, complicate access for service and maintenance, and may exceed the building height limitation of the City of Morro Bay Code.

Civil Sitework

Site Preparation. The site will likely require significant soil improvements for construction of the new WWTP facilities. The site is subject to settlement from the loading produced by fill and new structures and from seismic events. The site must also be protected from flooding by the Morro Creek watershed during major storm events.

Fugro West, Inc. has prepared a Geotechnical Study dated May 5, 2010 that is attached as Appendix B. The Geotechnical Study summarizes the geotechnical conditions of the site based on review of existing geotechnical reports, soils borings, and field testing conducted for past projects. The Geotechnical Study will also serve as the basis to develop a field program (soil borings and tests) to be conducted when final design commences and used to prepare a Geotechnical Report that will specifically address the WWTP Upgrade Project.

The initial key findings of the Geotechnical Study were prepared using existing geotechnical data available for the site from previous geotechnical studies and as-built construction records. Using this information, the main geotechnical considerations that will need to be addressed for the project are summarized as follows:

- Groundwater has been encountered at depths ranging from 7 to 13 feet below existing grade.
- The design seismic event is estimated to have a peak ground acceleration of approximately 0.34g based on procedures defined by the Building Code, about two times that of 2003 San Simeon Earthquake.
- Seismic events have the potential to cause liquefaction of underlying sand layers and could result in settlements of approximately 4 inches or more.
- Placement of fill to raise the new plant site above the 100-year flood elevation may cause settlement from 3 to 9 inches of underlying unconsolidated soils.
- The soil conditions encountered in the previous explorations of the site are heterogeneous, and the potential for settlement or liquefaction hazards to impact the site varies considerably over the footprint of the site. The estimated seismic and static settlements range from essentially none to the estimated amounts presented above.
- A range of soil mitigation approaches, structural foundation systems, or combination thereof that will prevent excessive damage to the new treatment facilities from a seismic event will be investigated during the final design phase of the Project.
- The geotechnical study conducted during final design will need to provide additional field exploration, laboratory testing, and engineering analysis to further characterize these considerations for the design and construction of the project, and as a basis to recommend suitable mitigations for seismic hazards.

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The Wallace Group has prepared a report entitled "Wastewater Treatment Plant Flood Hazard Analysis" dated August 7, 2009 that concluded the existing WWTP site was subject to inundation from the Morro Creek watershed during a 100-year storm event to depths ranging from 3 to 4.5 feet. The site for the new WWTP will be developed with the placement of engineered fill to raise the new site above the 100-year flood elevation. Based on the current evaluation this will require raising the existing grade by approximately 5 feet to a nominal Elevation of 20 feet. This would place the grade of the new WWTP site approximately 1 foot above the predicted 100-year flood elevation.

Facility Layout. A conceptual plan of the new WWTP layout showing the relative location, size, and orientation of the WWTP facilities is shown on **Figure 3**. Note that the size, location, and orientation of the facilities shown are subject to further refinement during the final design phase of the Project. The conceptual layout shown is intended to accomplish the following objectives:

- Position public-access facilities (Operations Building and Household Hazardous Waste) at the "front face" of the new WWTP.
- Position employee-access facilities (Operations Building, Maintenance Building, and parking) in close proximity.
- Direct public-access traffic to primary access road.
- Consolidate residuals handling (screenings, grit, and sludge) in one location.
- Centralize service truck traffic associated with residuals hauling (screenings, grit, and sludge), chemical and fuel deliveries, and operation and maintenance items (e.g., spare parts and lubricants) for the Residuals Facility, Standby Power, Maintenance Building, and Chemical Station.
- Direct service truck traffic to secondary access road.
- Provide simplified, U-shaped flow path from existing influent sewer to Influent Pump Station to Residuals Facility to Oxidation Ditches to Secondary Clarifiers to Tertiary Filter to Chlorine Contact Basin to the existing outfall.
- Design Tertiary Filter and Chlorine Contact Basin for future addition of Title 22 Reclaimed Water capacity.

Landscaping. Landscaping for the interior grounds of the new WWTP site is expected to be minimal to reduce maintenance. The interior will generally be paved or rocked to accommodate routine truck traffic and occasional use of cranes around the perimeter of treatment facilities for operation and maintenance functions.

The perimeter of the new WWTP site is expected to be fenced for security and landscaped with trees, bushes, and or vines to provide a degree of natural screening of the WWTP plant interior from public view. The perimeter landscaping is intended to be low maintenance after the plantings are established.

Demolition of Existing WWTP. After construction of the new WWTP, the existing WWTP will be demolished to maximize the available space for the floodway. The demolition of the existing WWTP is intended to be part of the WWTP Upgrade Project and can begin after beneficial use of the new WWTP after startup.

MBCSD will conduct a hazardous material survey of the existing WWTP to determine if any materials such as asbestos, lead, or mercury are present and will require special handling and removal. Any such requirements will be included in the Contract Documents (Specifications and Drawings) for the WWTP Upgrade Project.

The following items regarding demolition of the existing WWTP that will be addressed in the Contract Documents include the following:

- Any equipment or materials desired for salvage by MBCSD.
- Depth below existing grade that existing facilities will be removed – likely in the range of 3-feet to 5-feet in depth.
- Underground structures below the depth of removal indicated above will be filled with engineered fill.
- Underground conduits to be filled with sand or grout – likely in the range of 8-inch to 12-inch diameter and larger.
- Surface treatment of site after demolition – either pavement or rock.

Household Hazardous Waste Station. The existing WWTP provides space for a household hazardous waste (HZ) station operated by the Integrated Waste Management Authority. The design intent for the new WWTP Upgrade Project is to continue to provide space and vehicle access for this facility. The station will be placed on engineered fill to raise the facility above the 100-year flood elevation. Due to space constraints the placement of engineered fill and the relocation of the HZ Station will be deferred until the second phase of the WWTP Upgrade Project construction after the existing WWTP facilities are demolished. Refer to **Figure 3** – Conceptual Site Plan for the proposed location of the HZ Station.

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Architectural

General. The architectural treatment for the Operations Building, Maintenance Building, and Residuals Facility will provide a consistent design theme that will be compatible with the surrounding site and with other elements of the WWTP Upgrade Project. Possible exterior treatment elements include reinforced concrete, concrete masonry block, or some combination of the above. The benefits of selecting concrete-based finish materials include visual appeal, cost effectiveness, low maintenance, and resistance to corrosion. Exterior ferrous metals will be avoided because of the marine salt air environment.

Operations Building. An Operations Building will be provided with administrative offices, laboratory, control room, locker rooms, break room, and conference room.

Refer to **Figure 6A** and **Figure 6B** for one example of exterior architectural treatment for the Operations Building utilizing the concrete-based materials discussed above. Note that the concept shown is presented for illustrative purposes and will be subject to changes and further refinement during the design phase.

Maintenance Building. A Maintenance Building will house work space for repair and servicing treatment plant equipment. The work space will include work benches, storage racks for spare parts and tools, and storage cabinets for lubricants and related gear. A bridge crane will be included for lifting and moving equipment items for repair and service.

The Maintenance Building will also include an electrical room that will house switchgear, motor control centers, and variable frequency drives for the treatment plant equipment.

Refer to **Figure 7** for one example of exterior architectural treatment for the Maintenance Building utilizing the concrete-based materials discussed above. Note that the concept shown is presented for illustrative purposes and will be subject to changes and further refinement during the design phase.

Residuals Facility. The Residuals Facility is currently configured for outdoor exposure with a 3-sided wall (west, north, and east) that provides visual screening of the screenings, grit, and sludge handling equipment. The screening walls will also block prevailing winds from the oceanside to minimize exposure to the salt air. The open side (south) will facilitate access for equipment maintenance. The screening walls will be designed to be consistent with the architectural treatment of the Operations Building and the Maintenance Building.

Refer to **Figure 8A** and **Figure 8B** for one example of exterior architectural treatment for the Residuals Facility utilizing the concrete-based materials discussed above. Note that the concept shown is presented for illustrative purposes and will be subject to changes and further refinement during the design phase.

Treatment Facilities

The general treatment concept with pretreatment of influent wastewater, extended aeration with oxidation ditches and secondary clarifiers, tertiary filtration, effluent disinfection, and solids handling has been outlined above. A more detailed discussion of the individual treatment facilities consisting of plant processes and major equipment items is presented below. Refer to the conceptual site plan, plant flow schematic, and hydraulic profile shown in **Figure 3**, **Figure 4**, and **Figure 5**, respectively.

In many instances there are several equipment options available to perform a specific function for a given treatment process. Equipment items subject to further evaluation for selection will be noted in the descriptions below. The technical advantages and disadvantages along with associated costs of these equipment options will be evaluated in greater detail during the initial phase (Preliminary Design) of Final Design. The Design Team will work with MBCSD staff to select the options that provide the best value for the Project.

Influent Pump Station. The influent pump station will consist of submersible pumps located in a below-grade wet well. Multiple pumps with variable-speed drives will be provided to meet the expected range of influent flow with overlapping capacities to avoid gaps or steps in pumping delivery.

Residuals Facility. The Residuals Facility will include the following items:

- Mechanical fine screens
- Screenings conveyance
- Screenings washer/compactors

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- Grit removal unit
- Grit slurry pumps
- Grit washer
- Solids dewatering units
- Polymer handling and feed system

Screening equipment suitable for the Project include center-feed drum screens, in-channel drum screens, or center-feed band screens. Screenings conveyance can be accomplished with shaftless screw conveyor or with a sluicing trough. Screenings washer/compactors can be hydraulic or rotary screw units. The washed and compacted screenings will be discharged into a residuals bin for off-site disposal. Screening equipment options will be evaluated and selected during Preliminary Design.

A grit removal unit suitable for the Project includes a mechanical vortex grit unit or a multi-tray vortex grit unit. Grit washer systems include screw classifier with cyclones or swirl concentrators. Washed grit will be discharged to a residuals bin for off-site disposal. Grit removal equipment options will be evaluated and selected during Preliminary Design.

Solids dewatering equipment suitable for the Project include belt presses, centrifuges, and rotary screw presses. A polymer mixing and feed system will be provided to condition the feed sludge to the solids dewatering unit. Dewatered solids will be discharged to a roll-off bin for off-site processing and disposal. Solids dewatering equipment options will be evaluated and selected during Preliminary Design.

One preliminary configuration of the Residuals Facility has been developed to facilitate development of the conceptual site plan and conceptual hydraulic profile shown in **Figure 3** and **Figure 5**, respectively. Center feed drum screens, mechanical vortex grit unit, and rotary screw presses were selected for illustrative purposes as representative equipment for the screening, grit removal, and solids dewatering functions, respectively. Note that these choices were made for purposes of developing the conceptual site plan and conceptual hydraulic profile. The final selection of the appropriate equipment will be made during Preliminary Design as outlined above. The selection of alternative equipment as well as the refinement of the design as the Project progresses will likely modify the preliminary configuration shown.

The preliminary configuration of the Residuals Facility assumes that the attendant equipment is installed outdoors as shown in **Figure 8A** and **Figure 8B**. An upper floor will be required to support equipment and provide access for operation and maintenance. Three exterior sides (west, north, and east) of the proposed Residuals Facility will be provided with a full-height wall to provide protection from prevailing winds and to provide architectural treatment of sides that would be visible to the public. The south side facing the interior of the plant campus and the top of the elevated deck would be open to facilitate access for residuals and equipment removal.

Oxidation Ditches. Oxidation ditches utilize an extended aeration activated sludge process with secondary clarifiers to achieve excellent removals of BOD, TSS, and NH3 and to produce a high quality effluent suitable for tertiary filtration. The extended aeration will be supplemented with an anaerobic zone to produce a better settling sludge for reliable secondary clarifier performance and an anoxic zone to reduce aeration demand and restore alkalinity. The oxidation ditch technology provides reliable, robust biological treatment, can readily handle peak flows and loadings, and is easy to operate and maintain.

Aeration systems suitable for the Project include the following:

- Deck-mounted vertical-shaft surface aerators to supply the oxygen demand for treatment and the mixing energy to keep the mixed liquor solids in suspension.
- Fine-bubble diffusers installed at the floor of the oxidation ditch to supply the oxygen demand for treatment and submersible horizontal mixers that provide the mixing energy to propel the mixed liquor in the "racetrack" of the oxidation ditch basin. Air for the fine-bubble diffusers is provided by aeration blowers that would be located adjacent to the Oxidation Ditches.

The aeration system options for the Oxidation Ditches will be evaluated and selected during Preliminary Design. The major considerations in this evaluation will be the energy savings afforded by higher aeration efficiency of the fine-bubble system versus the higher initial cost of the fine-bubble system with blowers and the ease of operation and maintenance of the surface aerators.

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Secondary Clarifiers. Secondary Clarifiers are circular tanks that provide gravity separation of the biological solids (mixed liquor) produced by the Oxidation Ditch. The biological solids settle and form a sludge blanket in the lower zone of the clarifier. The clarified effluent forms a clear water zone above the sludge blanket. The clarified or secondary effluent overflows effluent weirs and is routed to the Tertiary Filter and Chlorine Contact Basin for further treatment. The sludge in the lower zone settles and is withdrawn by pumps to the head end of the Oxidation Ditches as return activated sludge (RAS). The RAS delivers the biological solids from the Secondary Clarifiers back to the Oxidation Ditches for treatment of the influent wastewater.

A portion of the settled sludge is removed from the activated sludge system at periodic intervals as waste activated sludge (WAS). The WAS is pumped to the solids dewatering system at the Residuals Facility. The RAS and WAS pumps will be located between the Secondary Clarifiers.

Secondary Pump Station. The Secondary Pump Station will be provided to lift the secondary effluent from the Secondary Clarifiers to the Tertiary Filter and Chlorine Contact Basin. The supplemental pumping provided by the Secondary Pump Station is anticipated to be necessary to avoid excessive height for the upstream facilities, in particular the Residuals Facility. The need for the Secondary Pump Station will be evaluated and verified during Preliminary Design as previously mentioned.

Tertiary Filter. A tertiary filtration system will be provided with capacity that matches the PSDW average flow of the WWTP. The Tertiary Filter is anticipated to be a cloth filter or equivalent unit that will provide a high degree of suspended solids removal and would be suitable to produce Title 22 Reclaimed Water in the future.

Chlorine Contact Basin / Chemical Station. A Chlorine Contact Basin (CCB) similar in function to the existing unit will be provided for disinfection of the treated effluent. The CCB will be designed with suitable length to width ratio for effective disinfection and provided with two chambers to facilitate operation and maintenance functions. The Chlorine Contact Basin will be designed to accommodate future Title 22 Reclaimed Water production.

A Chemical Station with provisions for storage and feeding of sodium hypochlorite (liquid bleach) for disinfection and sodium bisulfite for dechlorination at the Chlorine Contact Basin discharge will be provided. Chemical storage and feed equipment similar to existing equipment will be furnished.

A Utility Water (UW) Station will be provided to deliver treated effluent for ancillary water demands such as spray water and seal water.

Future Reclaimed Water. Capability for future reclaimed water facilities to meet Title 22 requirements will be included in the Project. Provisions for an additional tertiary filter and reclaimed water pump station will be included with the design. Refer to the technical memorandum to MBCSD entitled "TF/CCB/Title 22 Considerations" dated June 16, 2010 and prepared by MWH that is attached as Appendix C for additional information regarding provisions for future reclaimed water and sizing of the Tertiary Filter and Chlorine Contact Basin units.

Support Utilities

Potable Water. Potable water will be provided by extending the existing water main in Atascadero Road as a loop in and out of the new plant site using the public and service access road corridors. Connections for potable water uses within the plant will be protected with backflow preventers. The potable water loop will be provided with hydrants for on-site fire protection.

Natural Gas. Natural gas for building and water heating will be provided by connection to the existing gas main in Atascadero Road.

Electrical Power. Electrical power will be provided with a new electrical service from PG&E to serve the new WWTP. The new electrical service will include transformer and metering equipment.

Standby Power. A diesel engine-generator will be provided for standby power to operate the new WWTP in the event of an electrical power outage. The engine-generator set will be provided in a self-contained outdoor enclosure with fuel tank. The fuel tank will also be used for fueling WWTP vehicles.

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Temporary Solids Handling

Temporary solids handling facilities will be required to provide sludge dewatering during construction of the new WWTP. The new WWTP will be located in the portion of the site that contains the existing sludge drying beds. The sludge drying beds will be demolished for placement of engineered fill to raise the site above the 100-year flood elevation and to support the new treatment facilities.

During construction of the new facilities the existing WWTP will continue to operate and generate digested sludge. In lieu of routing the digested sludge to the existing sludge drying beds, the digested sludge will be dewatered with temporary sludge dewatering equipment and hauled off-site for disposal.

At this time it is anticipated that MBCSD will furnish and operate the temporary solids handling facilities. Two approaches that will be evaluated during the initial phase (Preliminary Design) of Final Design are the following:

- Leasing of temporary sludge dewatering equipment.
- Prepurchase of sludge dewatering equipment that would subsequently be relocated at the Residuals Facility for permanent installation as part of the WWTP Upgrade.

Project Schedule

RWQCB Conversion Schedule. The Regional Water Quality Control Board (RWQCB) and MBCSD have agreed to a Conversion Schedule to achieve full secondary treatment with the WWTP Upgrade Project. Key milestones in the Conversion Schedule for the design and construction of the WWTP Upgrade Project are listed in Table C.

Table C – Conversion Schedule Design and Construction Milestones

Task	Milestone Date
Initiate Design	Sep 30, 2010
Issue NTP for construction	May 29, 2012
Complete construction and commence startup	Jan 31, 2014
Commissioning (Compliance with secondary treatment)	Mar 31, 2014

EIR and Permits. The Environmental Impact Report (EIR) for the WWTP Upgrade Project is being prepared by ESA. The EIR will be coordinated with the final Facility Master Plan consisting of the Draft FMP, Draft FMP – Amendment No. 1, and the FMP – Amendment No. 2.

Permits that are anticipated to be obtained for the WWTP Upgrade Project include the following:

- City of Morro Bay Building Permit
- Californian Coastal Development Permit
- San Luis Obispo County APCD Permit for Standby Power
- Stormwater Pollution Prevention Plan

Project Schedule. The current Project Schedule is shown in **Figure 9**. The objective of the Project Schedule is to complete the remaining tasks in accordance with the Conversion Schedule milestone dates.

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Sequence of Construction. The sequence of construction for the WWTP Upgrade Project is relatively straightforward, but must be conducted to minimize disruption to ongoing operation and maintenance activities at the existing WWTP. The preliminary list of critical activities that will be identified and described with project constraints in the Contract Documents (Specifications and Drawings) include the following:

- Temporary solids handling equipment.
- Site preparation for new WWTP site.
- Placement of engineered fill for new WWTP site.
- Construction of new WWTP Upgrade facilities.
- Startup and commissioning of WWTP Upgrade facilities.
- Demolition of existing WWTP.
- Placement of fill and sitework for new parking lot and relocation of the Household Hazardous Waste Station.

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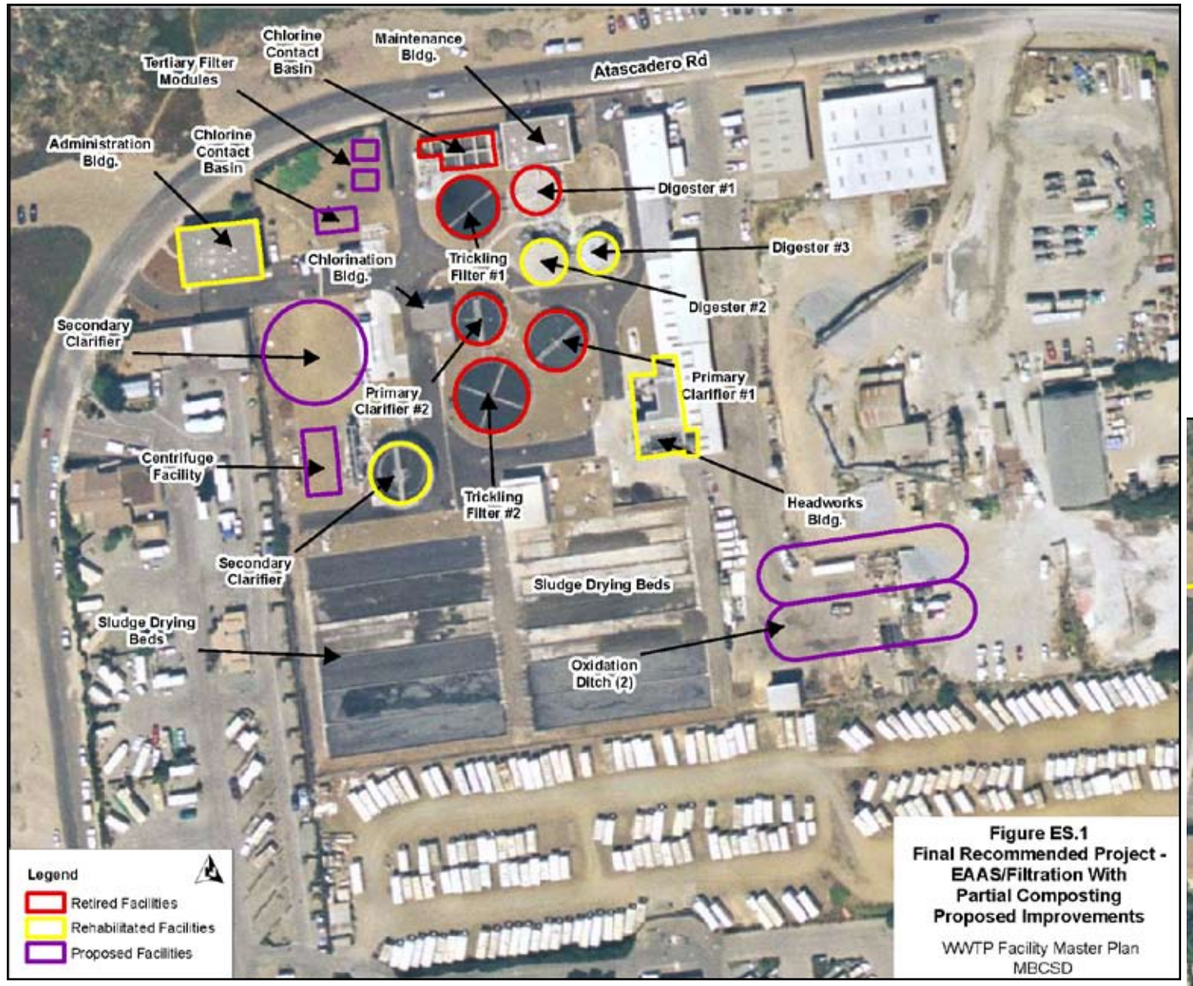


FIGURE 1
DRAFT FMP SITE PLAN

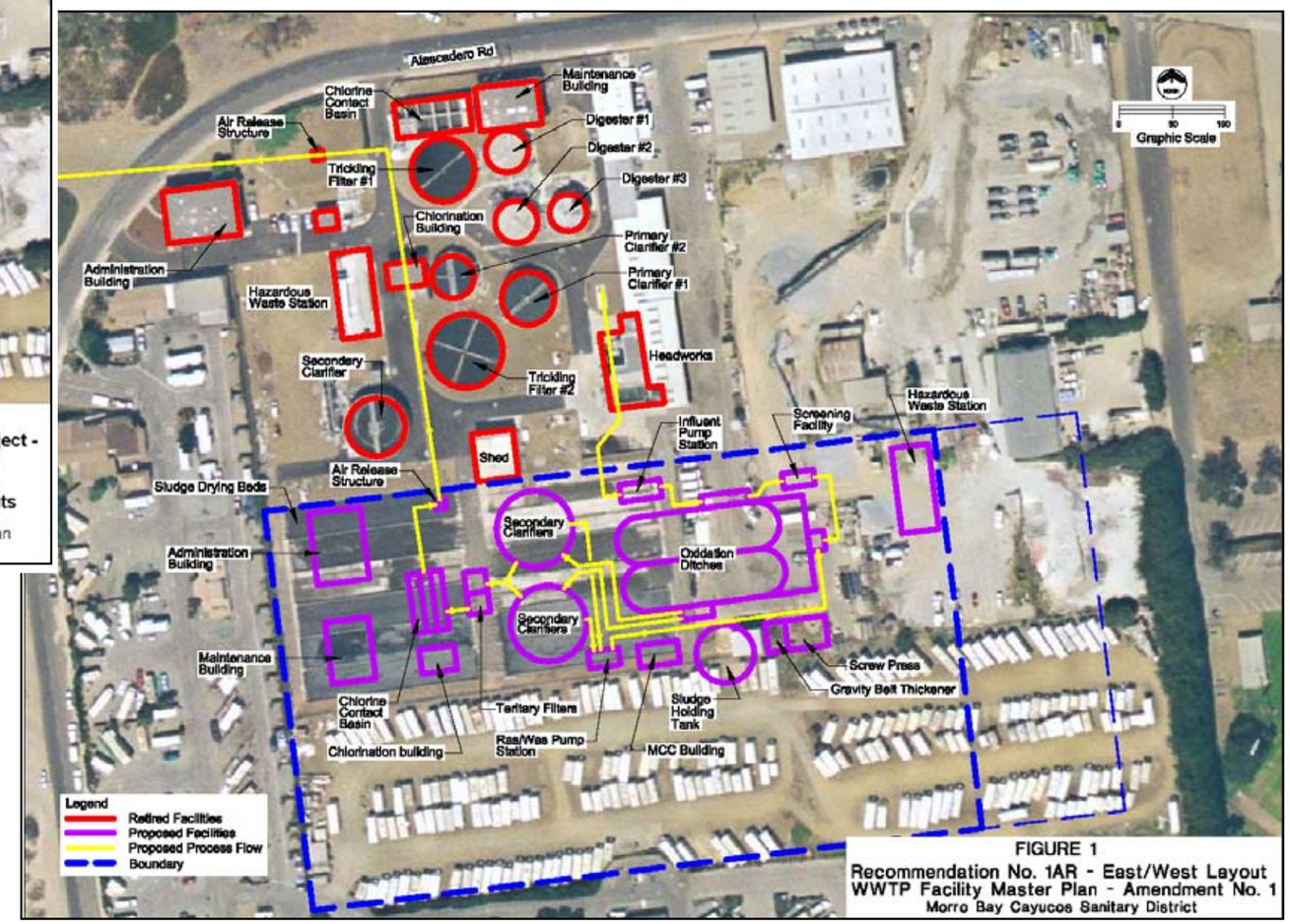
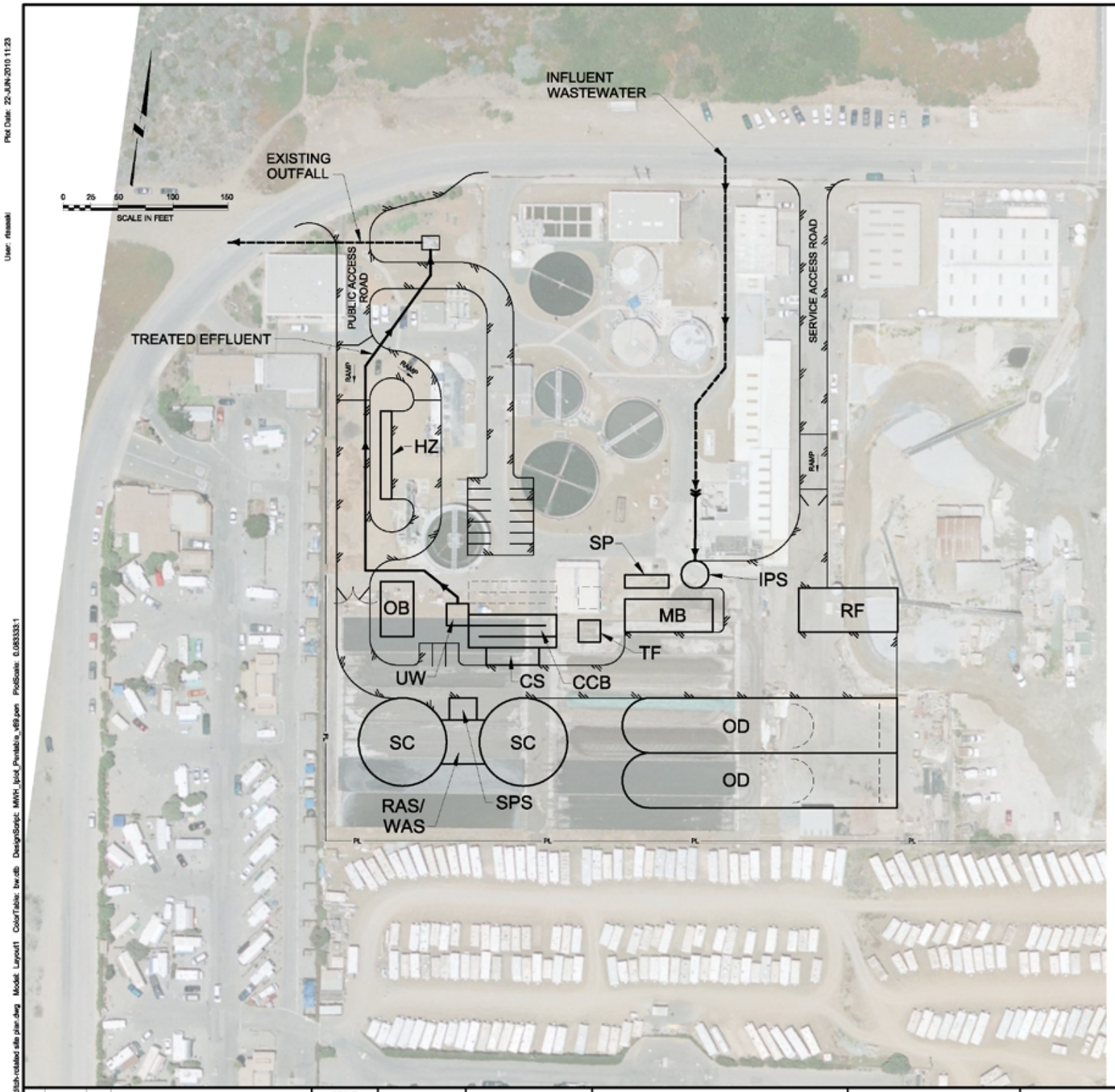


FIGURE 2
DRAFT FMP - AMMENDMENT NO. 1 SITE PLAN



LEGEND

CS	CHEMICAL STATION
CCB	CHLORINE CONTACT BASIN
HZ	HOUSEHOLD HAZARDOUS WASTE
IPS	INFLUENT PUMP STATION
MB	MAINTENANCE BUILDING
OB	OPERATIONS BUILDING
OD	OXIDATION DITCHES
RAS	RETURN ACTIVATED SLUDGE
RF	RESIDUALS FACILITY
SC	SECONDARY CLARIFIERS
SP	STANDBY POWER
SPS	SECONDARY PUMP STATION
TF	TERTIARY FILTER
UW	UTILITY WATER
WAS	WASTE ACTIVATED SLUDGE

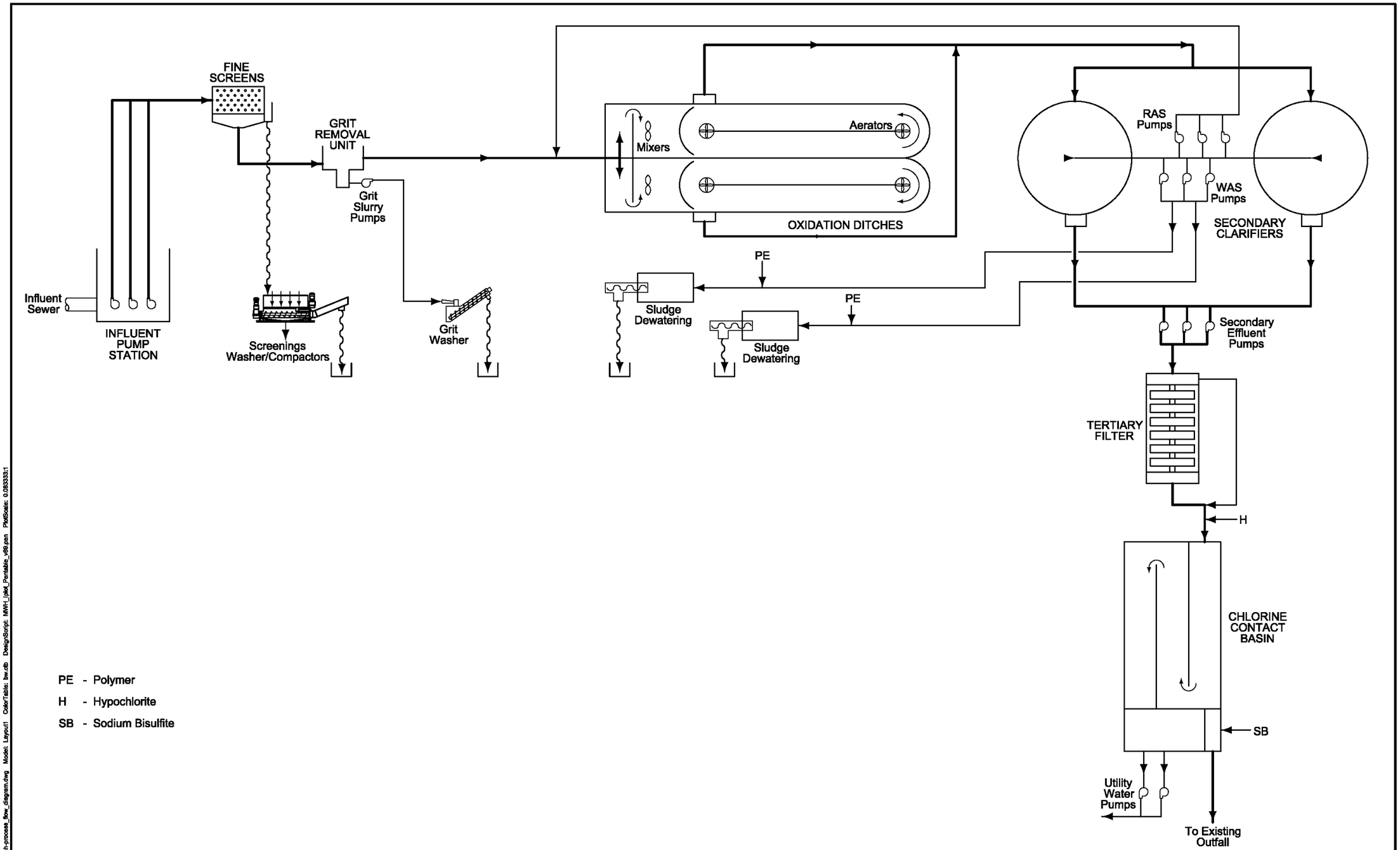
REV	DATE	BY	DESCRIPTION

SCALE 1"=100'

DESIGNED S.HYLAND
DRAWN R.SASAKI
CHECKED

MWH

FIGURE 3
CONCEPTUAL SITE PLAN



REV	DATE	BY	DESCRIPTION

SCALE
NO SCALE

DESIGNED S HYLAND
DRAWN D MASON
CHECKED

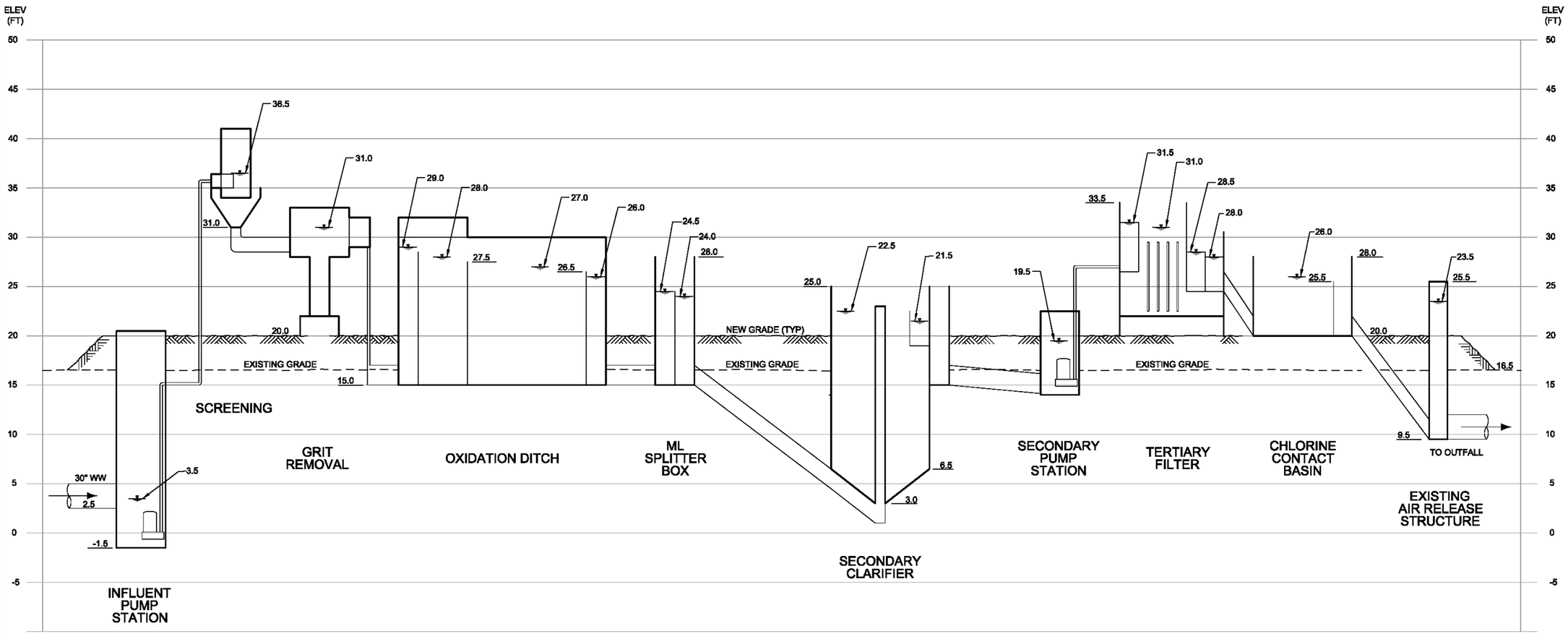
MWH

FIGURE 4
CONCEPTUAL FLOW SCHEMATIC

File: Oxidation Ditch-process_flow_diagram.dwg Model: Layout1 ColorTable: bw.ctb DesignScript: MWH_Ipd4_Portable_v98.pan PlotScale: 0.083333:1

GENERAL SHEET NOTES

1. DATUM FOR ELEVATIONS IS NAVD 88
2. CONCEPTUAL WATER SURFACE ELEVATIONS ARE SHOWN FOR PEAK DAY FLOW OF 5.6 MGD



REV	DATE	BY	DESCRIPTION

SCALE
VERT: 1"=10'

DESIGNED S HYLAND
DRAWN L BRADY
CHECKED

MWH

FIGURE 5
CONCEPTUAL HYDRAULIC PROFILE

File: oxidation ditch-hydraulic_profile_model.dwg Model: Layout1 ColorTable: bw.ctb DesignScript: MWH_Iplot_Printable_v69.pen PlotScale: 0.083333:1 User: rtasaki Plot Date: 15-JUN-2010 07:37

FIGURE 6A
OPERATIONS BUILDING ENTRY

FIGURE 6B
OPERATIONS BUILDING REAR

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FIGURE 7
MAINTENANCE BUILDING

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FIGURE 8A

FIGURE 8A
RESIDUALS FACILITY

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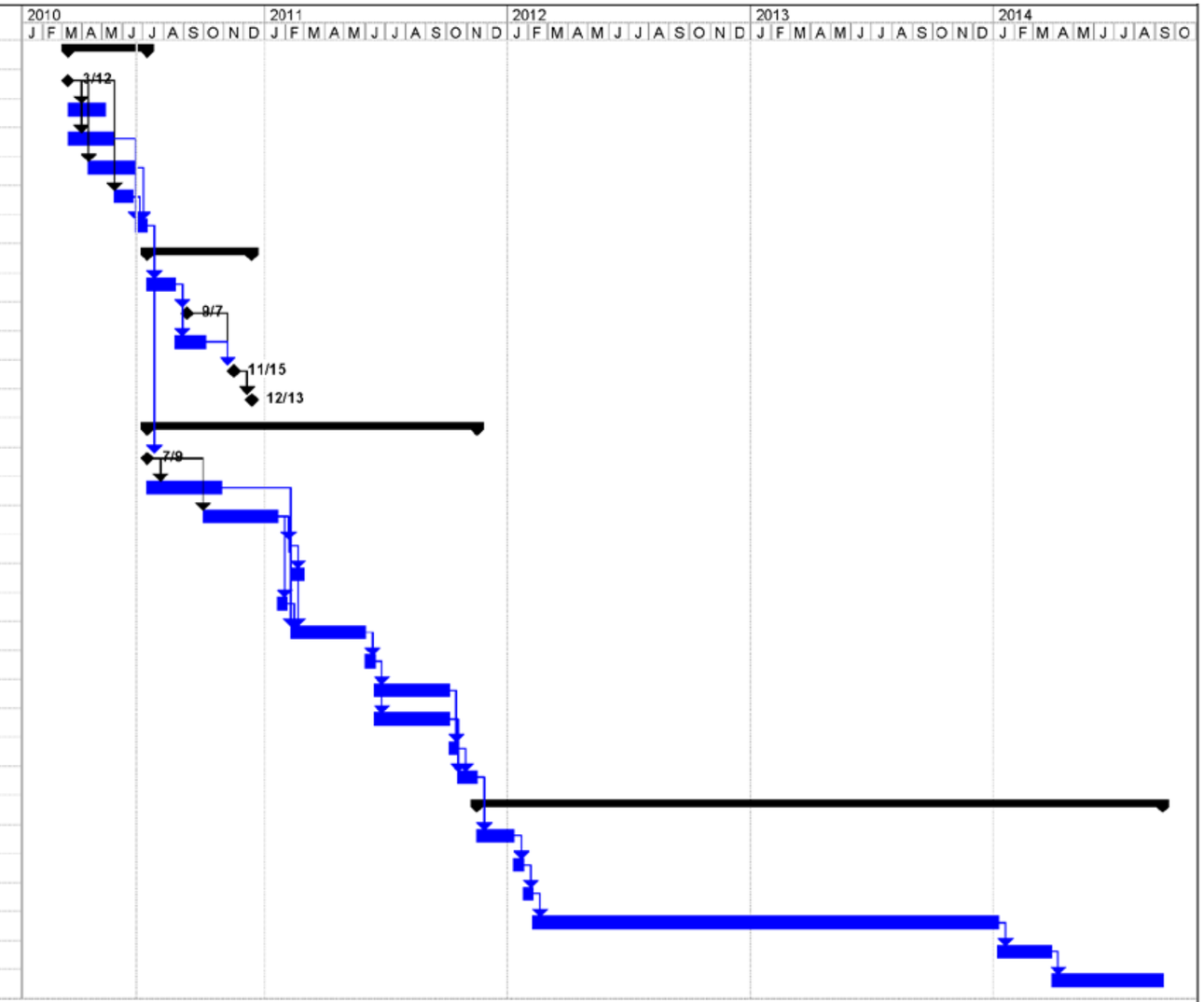
FIGURE 8A
RESIDUALS FACILITY REAR

ID	Task Name	Duration	Start	Finish	Predecessors
1	**Project Report Phase**	**85 days**	**Fri 3/12/10**	**Thu 7/8/10**	
2	Project Report NTP	0 days	Fri 3/12/10	Fri 3/12/10	
3	Geotechnical Study	39 days	Fri 3/12/10	Wed 5/5/10	2
4	FMP - Amendment No.2 Draft	49 days	Fri 3/12/10	Wed 5/19/10	2
5	Revised Flows and Loadings TM	51 days	Mon 4/12/10	Mon 6/21/10	2
6	TF/CCB/Title 22 TM	19 days	Fri 5/21/10	Wed 6/16/10	2
7	FMP - Amendment No.2 Final	13 days	Tue 6/22/10	Thu 7/8/10	6,4,5
8	**Environmental and Permit Phase**	**111 days**	**Fri 7/9/10**	**Mon 12/13/10**	
9	Resume Public Draft EIR	30 days	Fri 7/9/10	Thu 8/19/10	7
10	Draft EIR Public Hearing	0 days	Tue 9/7/10	Tue 9/7/10	9
11	Public Draft EIR comment period	32 days	Fri 8/20/10	Mon 10/4/10	9
12	Planning Commission CUP/CDP/Final EIR review	0 days	Mon 11/15/10	Mon 11/15/10	11,10
13	City Council EIR certification & CUP/CDP approval	0 days	Mon 12/13/10	Mon 12/13/10	12
14	**Final Design Phase**	**354 days**	**Fri 7/9/10**	**Wed 11/16/11**	
15	Final Design NTP	0 days	Fri 7/9/10	Fri 7/9/10	7
16	Draft Geotechnical Report	80 days	Fri 7/9/10	Thu 10/28/10	15
17	Preliminary Design Report-Draft	80 days	Fri 10/1/10	Thu 1/20/11	15
18	Value Engineering	3 days	Mon 2/7/11	Wed 2/9/11	17FS+11 days
19	Preliminary Design Report-Final	14 days	Thu 2/10/11	Tue 3/1/11	18
20	MBCSD Preliminary Design - comments	10 days	Fri 1/21/11	Thu 2/3/11	17
21	60% Design Submittal	80 days	Thu 2/10/11	Wed 6/1/11	20,16,18
22	MBCSD 60% Design - comments	10 days	Thu 6/2/11	Wed 6/15/11	21
23	90% Design Sumbittal	80 days	Thu 6/16/11	Wed 10/5/11	22
24	Final Geotechnical Report	80 days	Thu 6/16/11	Wed 10/5/11	22
25	MBCSD 90% Design - comments	10 days	Thu 10/6/11	Wed 10/19/11	23
26	Bid Documents	20 days	Thu 10/20/11	Wed 11/16/11	24,25
27	**Bid/Construction Phase**	**737 days**	**Thu 11/17/11**	**Fri 9/12/14**	
28	Advertise/Receive Bids	40 days	Thu 11/17/11	Wed 1/11/12	26
29	Award of Contract	10 days	Thu 1/12/12	Wed 1/25/12	28
30	Construction NTP	10 days	Thu 1/26/12	Wed 2/8/12	29
31	Construction of new plant	500 days	Thu 2/9/12	Wed 1/8/14	30
32	Plant start-up and Commissioning	57 days	Thu 1/9/14	Fri 3/28/14	31
33	Demolition of existing plant	120 days	Mon 3/31/14	Fri 9/12/14	32



Project: Overview Project Schedule_M
Date: Tue 6/22/10

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

Overview Project Schedule_Morro Bay WWTP_100622_rev1_msm.mpp Page 1 4/2/10

FIGURE 9
PROJECT SCHEDULE

APPENDIX A

Revised Flows and Loadings TM

MEMORANDUM

MWH

To:	Bruce Keogh Bill Callahan Dylan Wade	**Date:**	June 21, 2010
From:	Steve Hyland	**Client:**	MBCSD
Subject:	MBCSD WWTP Upgrade Revised Flows and Loadings	**Reference:**	1008613.020101

Introduction

The influent wastewater flow and loadings presented in the draft FMP were established for a combined MBCSD build-out population that would be reached by Y2021 for the City of Morro Bay (MB) and Y2015 for the Cayucos Sanitary District (CSD). These flow and loading projections were based on historical record information gathered for a 5-year period from Y2002 through Y2006 at the time the draft FMP was prepared.

MBCSD expressed concern that this period of record is too limited for the following reasons:

- Flow and loading data are now available for Y2007 to the present.
- The period of record for Y2002 – Y2006 were relatively dry years.

Consequently, flow data from Y1995 through Y2009 (15 years) were gathered and evaluated to establish the basis of design for flow parameters. Loading data for BOD and TSS for the period from Y2002 through Y2009 (8 years) were also gathered and evaluated. The results of this re-evaluation are presented below.

Existing Flow Data

The daily flow data were analyzed for the period of record from January 1995 through December 2009. The flow values reported for the period prior to August 2001 are based on effluent flow measurements with a propeller flow meter installed at the inlet to the chlorine contact basin. The flow values from August 2001 forward are based on influent flow measurements with a Palmer-Bowlus flume installed in the influent sewer upstream of the existing influent pump station. The influent flow measurements are more accurate and have been used for reporting to the Regional Water Quality Control Board (RWQCB) since the Palmer-Bowlus flume was installed.

The recorded effluent flow values from the propeller meter are consistently higher than the recorded influent flow values from the Palmer-Bowlus flume. Daily flow data for Y2005 were obtained for comparison of the influent Palmer-Bowlus flume and the effluent propeller meter. Y2005 was selected as having the highest range of flows for any annual period after August 2001 when the Palmer-Bowlus flume was installed.

The effluent flow data was plotted against its companion influent flow data for Y2005 as shown in **Figure 1**. A good correlation was obtained with a linear best-fit equation as follows:

Plant Flow (mgd) = [Effluent Flow (mgd) – 0.231 mgd] / 1.046

Consequently, all plant flow readings prior to August 2001 based on the effluent propeller meter were corrected with the above equation and used for the analysis of plant flows. Two approaches were then used to revaluate the corrected data as described below. Note that a different approach was used to evaluate the peak hour flow (PHF) that is separately described elsewhere in this memo.

The first approach was to evaluate the monthly flow data presented in the monthly reports. The average flow value for all months over the period of record was computed to establish the current annual average day flow (AADF). The average monthly flow value for the months of July and August over the period of record was analyzed to establish the current peak season dry weather (PSDW) average day flow and the PSDW maximum month flow. The PSDW period is important as it represents the highest loading to the WWTP due to the influx of visitors during the months of July and August. The highest monthly average flow value was selected for all the months over the period of record to establish the current average day maximum month flow (ADMMF) value.

The second approach was to evaluate the daily flow data and compute current average day flow and peak day flow for the period of record. The maximum value of the 30-day running average for the period of record was also computed to establish the current peak 30-day flow that corresponds to the peak month flow for comparison.

The results of the two approaches are presented in **Table A** along with the values presented in the draft FMP for comparison. The flow values for the longer period of record are generally greater in magnitude than the values presented in the draft FMP. This would be expected because the longer period of record includes annual periods of wet weather years with higher rainfall that would result in higher influent flows. The flow values based on the monthly approach and the daily approach are comparable and the selected values shown are the values considered the most representative of the given parameter.

Table A – Existing Flow Values

Parameter (a)	Unit	Draft FMP (b)	Monthly Values	Daily Values	Selected Values
Period of Record	---	Y2002 – Y2006	Y1995 – Y2009	Y1995 – Y2009	Y1995 – Y2009
AADF	mgd	1.15	1.24	1.25	1.25
ADMMF	mgd	1.39	2.66		2.66
Annual Peak 30-Day	mgd			3.55	3.55
Annual PDF	mgd			5.42	5.42
PSDWF, Average Day	mgd	1.25	1.32	1.32	1.32
PSDWF, Peak Month	mgd	1.35	1.70		1.70
PSDWF, Peak 30-Day	mgd			1.71	1.71
PSDWF, Peak Day	mgd			2.49	2.49

(a) AADF = Annual Average Day Flow
ADMMF = Average Day Maximum Month Flow
PDF = Peak Day Flow
PSDWF = Peak Season Dry Weather Flow
(b) From draft FMP Tables 3.5, 3.6, 3.7, and 3.8.

Projected Flows

The selected flow values for existing conditions from **Table A** were then used to calculate the design flows that would be projected to occur at build-out population. Three approaches were considered for the flow projections.

The first approach considered would follow the methodology used in the draft FMP. The annual average day flow was projected proportional to the population growth for MB and CSD. Flow projection factors were developed for each flow parameter that represented the average ratio of the particular flow parameter to the average annual day flow, i.e., ADMMF/AADF, PSDWF/AADF, and PHF/AADF for the period of record (Y2002 – Y2006). These flow project factors were then applied to the projected AADF flow value to determine the projected flow parameters. However, this approach does not appear correct for the following reasons:

- The flow projection factors should represent the maximum ratio for the period of record, not the average ratio, to be representative of the peak conditions that could be expected in the future.
- The flow projection factors assume that the annual infiltration and inflow (I&I) contribution increases proportionally to population growth. However, the increase in population is modest and will likely not result in any significant changes to the existing collection system because the anticipated population growth will be infill. Consequently,

although the baseline sanitary wastewater increases, there would likely be little, if any, change in I&I contributions.

Note that this approach assumes that the PSWF visitor population increases proportionally to the increase in resident population. This assumption is more difficult to judge, but if the current blend of full-time occupancy and part-time occupancy is the same at build-out, then this assumption would be reasonable.

The second approach considered would apply a multiplier based on the projected population at build-out. An initial build-out multiplier of 1.16 was calculated for the projected population for the City of Morro Bay and the Cayucos Sanitary District as shown in **Table B**.

Table B – MBCSD Build-Out Multiplier

Description	Morro Bay	Cayucos	MBCSD
Current Population	10,544 (a)	5,134 (c)	15,678
Est. Population At Build-out	12,500 (b)	5,730 (d)	18,230
Build-out Multiplier			1.16

(a) Morro Bay average population for Y2002 – Y2006 calculated from data presented in Table 3.2 from draft FMP.
(b) Morro Bay build-out population (Y2021) as listed in Section 3.3.1 and Table 3.11 from draft FMP.
(c) Cayucos average population for Y2002 – Y2006 calculated from average of 2,567 equivalent dwelling units (EDUs) presented in Table 3.4 from draft FMP and multiplied by 2.0 capita per EDU.
(d) Cayucos build-out population (Y2015) calculated from 2,865 equivalent dwelling units (EDUs) presented in Table 3.12 from draft FMP and multiplied by 2.0 capita per EDU.

The build-out multiplier was contemplated to be applied to all flow parameters as a conservative way to establish the design flow parameters. The use of the build-out multiplier is appropriate for projecting the AADF. However, this approach does not appear correct for the ADMMF, PSDWF, and PHF flow parameters for the following reason:

- The use of the build-out multiplier assumes that the annual infiltration and inflow (I&I) contribution increases proportionally to population growth. However, the increase in population is modest and will likely not result in any significant changes to the existing collection system because the anticipated population growth will be infill. Consequently, although the baseline sanitary wastewater increases, there would likely be little, if any, change in I&I contributions.

Note that this approach assumes that the PSWF visitor population increases proportionally to the increase in resident population. This assumption is more difficult to judge, but if the current blend of full-time occupancy and part-time occupancy is the same at build-out, then this assumption would be reasonable.

The third approach considered and adopted is based on the following assumptions:

- The projected AADF flow would be based on the build-out or residential population increase.
- The projected AADF increase due to build-out population growth would be added to all annual flow parameters (except PHF as separately discussed).
- No allowance for an increase in I&I due to build-out or residential population growth would be added to flow parameters. This assumption assumes that MB and CSD would continue to monitor and rehabilitate the existing collection systems to prevent deterioration that would allow an increase of I&I above current levels.
- An increase in PSDW flow parameters for visitor population contributions would be assumed to be proportional to residential population growth.

With this approach, the increase to existing annual flow values and existing PSDW flow values to determine design annual flow parameters and design PSDW flow parameters at build-out would be 0.21 mgd and 0.22 mgd, respectively, as shown in **Table C**.

Table C – Projected Flow Increase

Description	Morro Bay	Cayucos	MBCSD
Current Population (a)	10,544	5,134	15,678
Est. Population At Build-out (a)	12,500	5,730	18,230
Population Increase	1956	596	
Current Residential Contribution (b)	87 gpcd	65 gpcd (c)	
Future AADF Increase	0.170 mgd	0.039 mgd	0.21 mgd
Future PSDWF Increase (d)	0.180 mgd	0.041 mgd	0.22 mgd

(a) Refer to Table B.
(b) Refer to draft FMP Tables 3.9 and 3.10 for initial per capita values of 80 gpcd and 60 gpcd reported for MB and CSD, respectively. These values have been prorated by the ratio of 1.25 / 1.15 to reflect the increase in AADF presented in Table A for the longer period of record.
(c) Assumes 2 capita per EDU.
(d) The PSDWF increase is prorated by the ratio of 1.32 / 1.25 to reflect the increase in PSDWF – Average Day to AADF presented in Table A for the longer period of record.

The addition of the annual and PSDW flow increases to the selected flow values from Table A result in the design criteria shown in **Table D**. Design values from the draft FMP are also presented for comparison.

Table D – Design Flow Parameters

Parameter	Unit	Draft FMP (a)	Selected Values – Existing (b)	Build-Out Increase	Design Values
AADF	mgd	1.33	1.25	0.21	1.5
ADMMF	mgd	1.62	2.66	0.21	2.9
Annual Peak 30-Day Flow	mgd		3.55	0.21	3.8
Annual PDF	mgd		5.42	0.21	5.6
PSDWF, Average Day	mgd	1.47	1.32	0.22	1.5
PSDWF, Peak Month	mgd	1.62	1.70	0.22	1.9
PSDWF, Peak 30-Day	mgd		1.71	0.22	1.9
PSDWF, Peak Day	mgd		2.49	0.22	2.7

(a) From draft FMP Tables 3.11, 3.12, and 3.20.
(b) From Table A.

Peak Hour Flow

The peak hour flow value is difficult to determine because of limitations in the measurement of instantaneous flows and hydraulic constraints as described below. The peak flow value has been established by reviewing flow data recorded during peak day events and evaluation of the effective flow capacities of the existing influent pumps, primary clarifiers, and existing outfall.

Continuous flow strip chart readings for selected days with the highest recorded peak day flows were retrieved from files and reviewed with plant staff. The selected days occurred during historical wet years prior to August 2001; so the strip chart readings are based on the effluent propeller meter. As previously discussed the effluent propeller flow meter values need correction to correspond to the influent Palmer-Bowlus flume. The effluent propeller meter is calibrated with a maximum flow of 5500 gpm or 7.9 mgd (uncorrected) which corresponds to 6.5 mgd (corrected). For the selected peak flow days there are significant hours when the readings are "pegged" at 7.9 mgd (uncorrected) indicating that actual flow exceeded 6.5 mgd (corrected). The highest peak hour flow would occur during these periods, but the magnitude cannot be directly quantified because the flow strip chart is truncated at the upper calibration limit.

The pumping capacity of the existing influent pump station (three pumps installed) was investigated by looking at pump curves and estimating a system curve. The nominal capacity of each existing pump based on the original certified pump curves is approximately 3.3 mgd. The

influent pump performance has declined over the years because of wear and tear on the pump impellers. A recent rehabilitation of one of the existing pumps is believed to have increased the rehabilitated pump's capacity by approximately 20 percent and close to its original capacity.

Although designed to operate with two duty and one standby pump, during large wet weather events, all three pumps are needed to meet the influent flow demand. If all three pumps were operated in parallel at full speed, a theoretical peak hour flow as high as (3 pumps @ 3.3 mgd each =) 9.9 mgd could be achieved based on the original pump curves. However, if the past performance of the pumps has declined during the period of record used for evaluation of flow, then the effective influent pump station capacity may be on the order of 8.0 mgd (assuming 20 percent capacity reduction).

The determination of peak hour flow is further complicated because during the large wet weather events, the operation of the influent pumps was manually controlled to prevent surcharging of the primary clarifiers. Under these conditions one of the three influent pumps would be temporarily decreased in speed by operator control to reduce the flow to the primary clarifiers. Drawing G5 – Hydraulic Profile in the drawings dated October 1982 prepared by Brown and Caldwell include Note 1 which reads: "A new primary effluent pump station will be required under Stage 2 improvements to discharge flows greater than 6.64 mgd." This note implies that at some flow rate greater than 6.64 mgd, the primary clarifiers will be surcharged.

When the influent pumps are manually operated to prevent surcharging of the primary clarifiers, the reduction in flow has been safely attenuated in the collection system. Without this attenuation an even higher peak flow would result, although the magnitude could not have been recorded (i.e., effluent flow meter pegged).

The hydraulic capacity of the existing outfall was investigated by reviewing three memos dated August 22, September 12, and September 30, 1991 prepared by City of Morro Bay Department of Public Works regarding the ocean outfall capacity. The results of the hydraulic analysis conducted by staff indicated that flow up to 8.0 mgd can be conveyed by the outfall without overtopping the existing air release structure. There is no record or reported observation that the air release structure has ever overflowed.

After review of the available information summarized above, the current peak hour flow is established at 8.0 mgd based on the following considerations:

- Reduction of effluent propeller meter flows by the correction factor previously discussed would reduce the "unseen" flow rate that is above the upper calibration limit of 6.5 mgd (corrected), but the magnitude of the corrected peak flow above 6.5 mgd is still unknown.
- The effective maximum pumping capacity of the influent pump station during the period of record for flow evaluation has likely been on the order of 8.0 mgd.
- The effective hydraulic capacity of the primary clarifiers before surcharging is expected to be 6.6 mgd or higher.
- The manual operation of the existing influent pumps during large wet weather events to prevent the primary clarifiers from surcharging reduces the peak flow reaching the plant by attenuation in the collection system.

- There is no indication that the nominal outfall capacity of 8.0 mgd has been exceeded.

The incremental flow increase of 0.2 mgd applied to other flow parameters has not been utilized for the peak hour flow parameter. The magnitude of peak hour flow is driven by the infiltration and inflow (I&I) in the collection system and is significantly greater than the sanitary wastewater flow. Given the inherent uncertainty of establishing the current peak hour flow parameter, the addition of the small flow increase of 0.2 mgd would not be relevant.

Existing Loading Data

The loading parameters were established by analyzing the BOD and TSS values for the period of record from January 2002 through December 2009. The influent wastewater is routinely analyzed for BOD and TSS concentrations (mg/L) with 24-hour composite samples obtained on a weekly basis. The BOD and TSS values were first screened for outliers by setting aside all values greater than three standard deviations of the average value for the period of record. The remaining values were used to calculate loadings by multiplying the concentration (mg/L) times plant flow (mgd) for the corresponding day times a conversion factor (8.34) to calculate daily loading (lb/day). The plant flows used to calculate loadings for the period prior to August 2001 based on the effluent propeller meter were corrected as previously discussed.

The annual average loading values and annual maximum day loading values were calculated from the daily loading values for the period of record. In a similar manner, the PSDW average day values and maximum day values were calculated for the months of July and August over the period of record.

The daily loading values were then ranked in ascending order and plotted as a percent of the total values that are less than the given value. These plots of BOD and TSS are shown in **Figure 2** and **Figure 3**, respectively. From these plots, which represent a version of probability charts, the current loading values for the Maximum 30-Day were obtained by extracting the values at the (335 day / 365 day =) 91.8 percentile.

The loading values from these analyses are presented in **Table E** along with the draft FMP values for comparison. Note that the annual average values and PSDW average day values from the draft FMP are somewhat higher than the values computed from the daily loadings for the same period of record. This difference is believed to result from the calculation of draft FMP loadings from average flow and average concentrations for each year rather than the average daily loadings for each year. This can be seen by the comparison of loadings calculated from the daily values versus the draft FMP values for the same Y2002 – Y2006 period of record. The loading values based on the daily approach over the Y2002 – Y2009 period of record were selected as the most representative to use for projecting the design loading parameters.

Table E – Existing BOD and TSS Loadings

Parameter	Unit	Draft FMP (a)	Daily Values (b)	Daily Values (b)	Selected Values
Period of Record	---	Y2002 – Y2006	Y2002 – Y2006	Y2002 – Y2009	Y2002 – Y2009
BOD					
Annual Average	lb/day	3,100	3,020	3,000	3,000
Annual Max 30-Day	lb/day			4,090	4,090
Annual Max Day	lb/day			5,540	5,540
PSDW, Average Day	lb/day	3,750	3,520	3,590	3,590
PSDW, Max Month	lb/day			4,720	4,720
PSDW, Max Day	lb/day			5,540	5,540
TSS					
Annual Average	lb/day	3,350	3,300	3,260	3,260
Annual Max 30-Day	lb/day			4,560	4,560
Annual Max Day	lb/day			6,090	6,090
PSDW, Average Day	lb/day	4,080	3,790	3,860	3,860
PSDW, Max Month	lb/day			5,410	5,410
PSDW, Max Day	lb/day			5,920	5,920

(a) The average values from draft FMP Tables 3.14 and 3.16 that appear to be calculated from average flow and average concentrations for each year.
(b) Daily values calculated from daily loadings.

Projected Loadings

The selected loading values for existing conditions from **Table E** were then used to calculate the design loadings that would be projected to occur at build-out population. Two approaches were considered for the loading projections.

The first approach considered would follow the methodology used in the draft FMP. Influent BOD and TSS concentrations were assigned for average annual and average PSDW conditions and multiplied by their respective projected average day flows to calculate the design loadings. However, as discussed above this approach does not appear correct because the use of average flow values and average BOD and TSS concentrations overstates the BOD and TSS loadings.

The second approach considered and adopted was to increase the current loading values by the build-out multiplier of 1.16 presented in **Table B** for annual conditions. This approach assumes that the BOD and TSS loadings are proportional to the resident population of the City of Morro Bay and the Cayucos Sanitary District. The resulting incremental increase in annual loadings is then added to the current PSDW loadings to establish the PSDW design loadings. This approach assumes that the visitor loading contributions during the PSDW period (July and August) also increase proportionally to resident population. The resultant design BOD and TSS loading values are shown in **Table F**.

Table F – Design Loadings

Parameter (a)	Unit	Draft FMP (b)	Selected Values (c)	Build-Out Increase	Design Values
BOD					
Annual Average	lb/day	3,800	3,000	480	3,500
Annual Max 30-Day	lb/day		4,090	650	4,700
Annual Max Day	lb/day		5,540	890	6,400
PSDW, Average Day	lb/day	5,000	3,590	570	4,200
PSDW, Max Month	lb/day		4,720	760	5,500
PSDW, Max Day	lb/day		5,540	890	6,400
TSS					
Annual Average	lb/day	4,000	3,260	520	3,800
Annual Max 30-Day	lb/day		4,560	730	5,300
Annual Max Day	lb/day		6,090	970	7,100
PSDW, Average Day	lb/day	5,200	3,860	620	4,500
PSDW, Max Month	lb/day		5,410	870	6,300
PSDW, Max Day	lb/day		5,920	950	6,900
TKN (d)					
Annual Average	lb/day				600
Annual Max 30-Day	lb/day				800
Annual Max Day	lb/day				1100
PSDW, Average Day	lb/day				710
PSDW, Max Month	lb/day				940
PSDW, Max Day	lb/day				1200

(a) PSDW = Peak Seasonal Dry Weather
(b) From draft FMP Table 3.20.
(c) From Table E.
(d) Note that TKN values are currently derived by assuming a TKN / BOD ratio of 0.17. Supplemental testing of influent wastewater samples will be conducted during Y2010 to obtain TKN data and modify the listed values if needed.

Summary

A summary of the design criteria developed for the WWTP Upgrade Project based upon the discussion above is presented in Table G.

Table G – WWTP Upgrade Design Criteria

Parameter (a)	**Unit**	**Design Values**
Flow		
AADF	mgd	1.5
ADMMF	mgd	2.9
Annual Peak 30-Day	mgd	3.8
Annual Peak Day	mgd	5.6
Peak Hour	mgd	8.0
PSDWF, Average Day	mgd	1.5
PSDWF, Peak Month	mgd	1.9
PSDWF, Peak 30-Day	mgd	1.9
PSDWF, Peak Day	mgd	2.7
BOD		
Annual Average	lb/day	3,500
Annual Max 30-Day	lb/day	4,700
Annual Max Day	lb/day	6,400
PSDW, Average Day	lb/day	4,200
PSDW, Max Month	lb/day	5,500
PSDW, Max Day	lb/day	6,400

TSS		
Annual Average	lb/day	3,800
Annual Max 30-Day	lb/day	5,300
Annual Max Day	lb/day	7,100
PSDW, Average Day	lb/day	4,500
PSDW, Max Month	lb/day	6,300
PSDW, Max Day	lb/day	6,900
TKN (b)		
Annual Average	lb/day	600
Annual Max 30-Day	lb/day	800
Annual Max Day	lb/day	1100
PSDW, Average Day	lb/day	710
PSDW, Max Month	lb/day	940
PSDW, Max Day	lb/day	1200

(a) PSDW = Peak Seasonal Dry Weather
(b) Note that TKN values are currently derived by assuming a TKN / BOD ratio of 0.17. Supplemental testing of influent wastewater samples will be conducted during Y2010 to obtain TKN data and modify the listed values if needed.

Please contact Steve Hyland if you have any questions.

c. Don Spiegel
Roger Stephenson

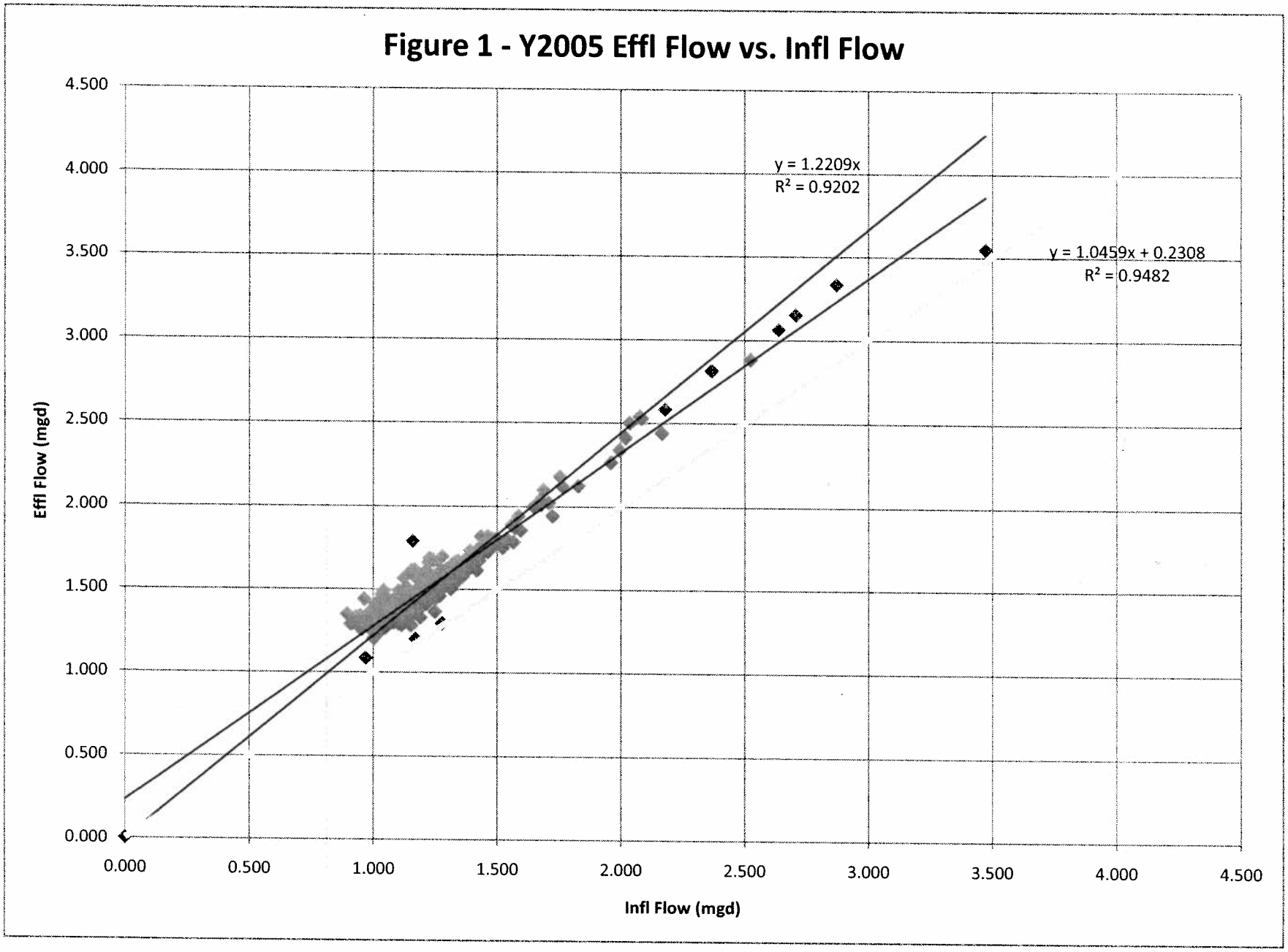


Figure 1 - Y2005 Effl Flow vs. Infl Flow
y = 1.2209x
R² = 0.9202
y = 1.0459x + 0.2308
R² = 0.9482
Effl Flow (mgd)
Infl Flow (mgd)
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500

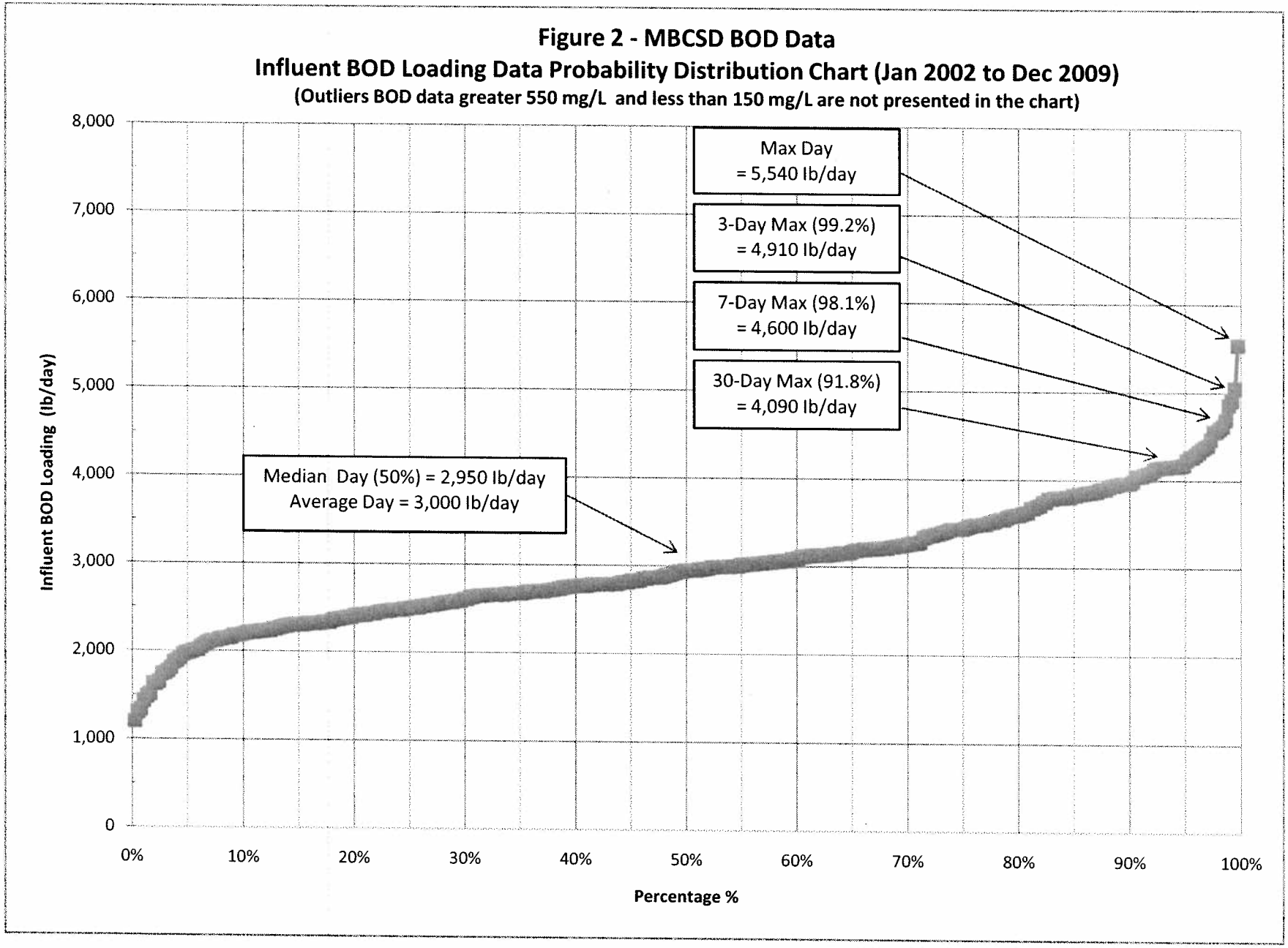


Figure 2 - MBCSD BOD Data
Influent BOD Loading Data Probability Distribution Chart (Jan 2002 to Dec 2009)
(Outliers BOD data greater 550 mg/L and less than 150 mg/L are not presented in the chart)
Max Day
= 5,540 lb/day
3-Day Max (99.2%)
= 4,910 lb/day
7-Day Max (98.1%)
= 4,600 lb/day
30-Day Max (91.8%)
= 4,090 lb/day
Median Day (50%) = 2,950 lb/day
Average Day = 3,000 lb/day
Influent BOD Loading (lb/day)
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
0%
10%
20%
30%
40%
50%
60%
70%
80%
90%
100%
Percentage %

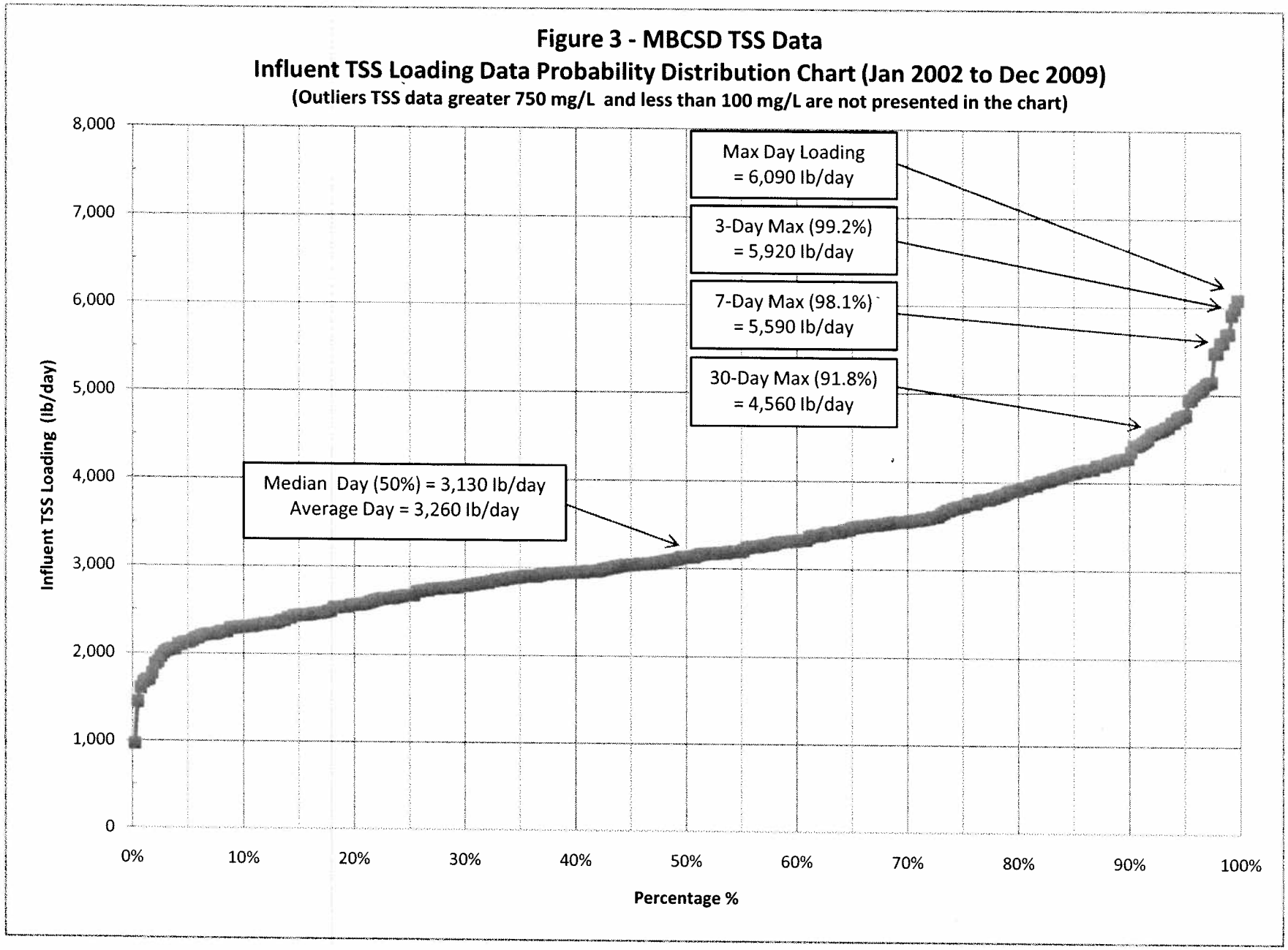


Figure 3 - MBCSD TSS Data
Influent TSS Loading Data Probability Distribution Chart (Jan 2002 to Dec 2009)
(Outliers TSS data greater 750 mg/L and less than 100 mg/L are not presented in the chart)
Max Day Loading = 6,090 lb/day
3-Day Max (99.2%) = 5,920 lb/day
7-Day Max (98.1%) = 5,590 lb/day
30-Day Max (91.8%) = 4,560 lb/day
Median Day (50%) = 3,130 lb/day
Average Day = 3,260 lb/day
Influent TSS Loading (lb/day)
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
0%
10%
20%
30%
40%
50%
60%
70%
80%
90%
100%
Percentage %

APPENDIX B

Geotechnical Study

FUGRO WEST, INC.

PRELIMINARY GEOTECHNICAL REPORT
MORRO BAY – CAYUCOS WASTEWATER TREATMENT PLANT UPGRADE
160 ATASCADERO ROAD
MORRO BAY, CALIFORNIA

Prepared for:
MWH Americas, Inc.

April 20, 2010

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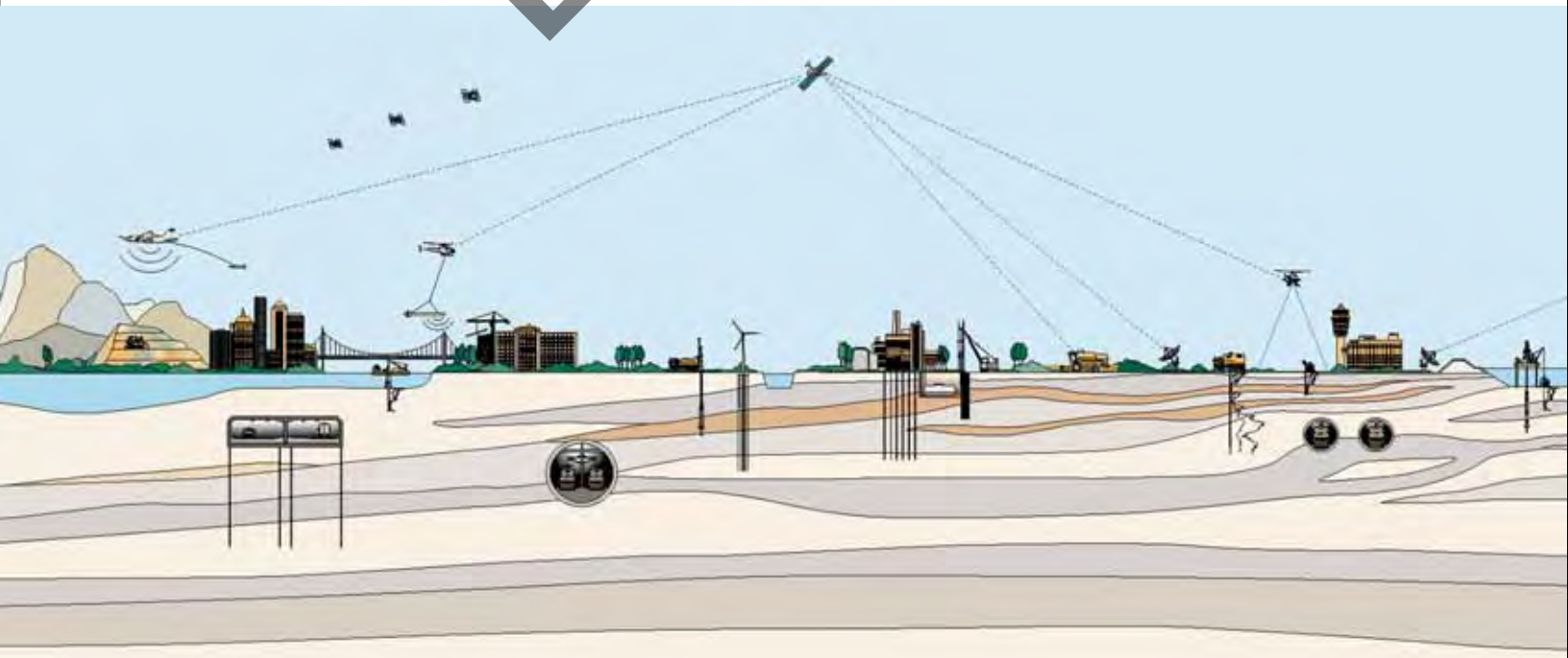

FUGRO WEST, INC.

660 Clarion Court, Suite A
San Luis Obispo, California 93401
Tel: (805) 542-0797
Fax: (805) 542-9311

April 20, 2010
Project No. 1445.010

MWH Americas, Inc.
2121 N. California Blvd, Suite 600
Walnut Creek, California 94596

Attention: Mr. Steve Hyland

Subject: Preliminary Geotechnical Report, Morro Bay – Cayucos Wastewater Treatment Plant Upgrade Project, 160 Atascadero Road, Morro Bay, California

Dear Mr. Hyland:

Fugro is pleased to submit this Preliminary Geotechnical Report for the Morro Bay – Cayucos Wastewater Treatment Plant Upgrade Project in Morro Bay, California. This report was prepared in accordance with the scope of services presented in our proposal dated October 29, 2009, and authorized under the consulting subcontract with MWH dated March 15, 2010.

The purpose of this report is to provide preliminary geotechnical considerations for design of the proposed upgrades to the wastewater treatment plant based on existing data from previous field explorations at the site. This report provides preliminary geotechnical considerations and opinions regarding site geology, soil and groundwater conditions encountered potential for the sites to be impacted by geologic hazards, and anticipated grading and foundation support for the proposed structures.

Please contact the undersigned if you have questions regarding this report, or require additional information.

Sincerely,

FUGRO WEST, INC.

Jonathan D. Blanchard, G.E. 2312
Principal Geotechnical Engineer

Chad Stoehr
Staff Engineer II

Copies: 4 – Addressee, 1 PDF

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Historical Aerial Photographs 1937 through 2005
Historical Topographic Maps 1900 to 1995

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1. PROJECT AND SITE DESCRIPTION

The project consists of upgrades to the existing Morro-Bay – Cayucos Wastewater Treatment Plant located at 160 Atascadero Road in Morro Bay, California. The site for the proposed improvements is located south of the existing wastewater treatment plant in the area of the existing sludge drying beds and adjacent land to the south and east. The location of the site relative to nearby streets and geographic landmarks is shown on Plate 1, Vicinity Map. The layout of the site, showing the preliminary layout of the proposed facilities is shown on Plate 2, Site Plan.

The main improvements associated with the project will include new administration and operation buildings, filters, oxidation ditches, and tank structures to replace the old facilities, improve treatment processes and withstand future demand. We expect the depth and height of below and above grade structures to range from approximately 5 to 20 feet below and 10 to 38 feet above existing grade, respectively. We understand from MWH that the current layout of the proposed facilities is subject to change as the project progresses.

The site is located approximately 1,000 feet east of the Pacific Ocean. The existing site grade ranges from approximately elevation (el.) 15 to 24 feet based on MB1 Exhibit 4.1 of the flood hazard analysis prepared by Wallace Group (2009). The terrain at the site vicinity is relatively flat lying ground near the mouth of Morro Creek located approximately 1,000 feet south of the site. Sand dunes and the beach border the site to the west. The site is bordered by a mobile home park to the west, trailer storage area to the south, Atascadero Road to the north, and Hanson's concrete plant to the east.

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2. WORK PERFORMED

2.1 PURPOSE

The purpose of this report is to provide preliminary geotechnical considerations and opinions for the proposed improvements to the wastewater treatment plant. The primary geotechnical considerations evaluated for the project are characterization of the subsurface materials, geologic hazards, anticipated site preparation and grading for support of the improvements, and foundation considerations. The information provided herein is preliminary and is not intended for design of the project. A subsequent design-level geotechnical study will be performed for design of the improvements.

2.2 SCOPE

To evaluate the geotechnical considerations for the project, we performed the following scope of work:

- Data review of existing plans and maps, subsurface data, geotechnical reports, construction records, and information obtained from our in-house files including published geologic maps, documents, and historical aerial photographs; and

- Preparation of this report summarizing the data obtained for the site, and our preliminary conclusions and recommendations regarding;
  - Geologic and seismic setting;
  - Soil and groundwater conditions encountered;
  - Potential for the site to be impacted by geologic hazards (such as strong ground motion, fault rupture, liquefaction, seismic settlement, landsliding, flooding, erosion, and expansive or collapsible soil conditions);
  - Need for mitigation of liquefaction, seismic settlement or other seismic related hazards, and a discussion of the types of mitigation that could be provided if needed;
  - Anticipated site grading and foundation support for the various structures;
  - Anticipated foundation types (such as piles or spread footings) for the various structures; and
  - Construction considerations relative to dewatering, excavation characteristics of the soils encountered, and need for shoring.

2.3 PREVIOUS STUDIES

As a basis for preparation of this report we reviewed previous reports, construction records and as-built information, and aerial photographs for the site. We also reviewed as-built plans, construction records, and a previous geotechnical report obtained from the City's archive records kept at the plant. Plant personnel assist in finding and obtaining these records from the archives.

2.3.1 Previous Reports

Geotechnical Consultants, Inc. (GTC) (1980) performed a design-level geotechnical investigation for previous improvements to the wastewater treatment plant. Subsurface data included standard penetration test (SPT) blowcounts obtained from 18-inch diameter bucket auger borings performed across the existing site. Earth Systems Pacific (ESP) (2009) performed a geotechnical feasibility study at the site as input to the Environmental Impact Report (EIR) for the proposed improvements. Field exploration for the study included hollow-stem auger borings and cone penetration testing (CPT). We were able to obtain the CPT data electronically, process the data in-house, and generate our own interpretation of the results. Logs of the CPT's generated by Fugro are included in Appendix A. During review of the ESP study, we observed that the boring information does not agree with the relative density (Dr) we estimated from the CPT data, and therefore we suspect the boring information and drilling methods used by ESP may not be reliable. Logs of the borings performed by GTC and ESP are available electronically and will be included in the design level report.

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2.3.2 Construction Records and As-Built Information

We reviewed construction records and as-built information from previous improvements to the plant in the early 1980's. The information included sub-soil stabilization and grading and paving plans (Brown and Caldwell, 1981), vibro-compaction reports (GKN Keller, 1983 and 1984), and confirmation and test borings performed during and after sub-soil stabilization (Pacific Geoscience, Inc. 1983 and 1984). The sub-soil stabilization was performed using deep compaction with stone columns. The limits of sub-soil stabilization performed in 1983 are shown on Plate 2, Site Plan. The stone columns were emplaced on a 6-foot triangular grid pattern, extended to el. 90 feet, and were intended to densify the soil to at least 75 percent relative density. The vibro-compaction reports included time and material quantity information at each stabilization location. A letter by GTC dated October 4, 1983 includes their evaluation of densification of the foundation soils after completion of the sub-soil stabilization and f verification borings. The letter includes a discussion that with the exception of one, all SPT test results had densities exceeding 75 percent relative density. Letters by GTC dated December 20, 1983 and November 1, 1984 indicate that subsoil stabilization was not performed beneath a portion of the existing administration building due to the presence of existing operations building with basement and the presence of dumped fill for the old headworks building. A system of grade beams was used to provide additional foundation support for the existing administration building. Additionally, the letters suggest that the sub-soil stabilization may have been deleted below the maintenance building, and the building was supported on a mat of compacted fill due to the timing and cost of the stabilization.

2.3.3 Historical Aerial Photographs and Topographic Maps

We also reviewed selected historical aerial photographs and topographic maps obtained from Environmental Data Resources (EDR 2010a, 2010b) that dated back to 1937 and 1900, respectively. The EDR historical aerial photographs and topographic maps are included in Appendix B. The following is a summary of the photographs reviewed as part of this study.

Summary of Historical Aerial Photograph Review

Date	Scale	Source	Key observations
1937	1" = 555'	Army	Site is rural and vacant. Adjacent land use is mainly dry farming and row crops. The location of the existing site is covered with sand dunes and vegetation. The mouth of Morro Creek is cut to north and is cut 100 to 200 feet into the ocean-side of the sand dunes immediately west of the site. Highway 1 appears to be 2-lanes.
1949	1" = 555'	Aero	Similar to 1937 photo except dunes appear mostly denuded of vegetation and active. Mouth of Morro Creek has retreated to south of the existing site extending east-west near current its location. Some land development occurring east of Highway 1.

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Date	Scale	Source	Key observations
1956	1" = 555'	Hycon	Improvements (possibly chlorine contact tank and biofilter) still evident at existing wastewater treatment plant are present at north end of site. Remaining site is still undeveloped. Unpaved access to the site terminates at plant. Appears site was constructed into the dune sands. Morro Creek fans west of the existing site. There is a moderate amount of development and new roads east of Highway 1.
1963	1" = 333'	Mark Hurd	Mouth of Morro Creek has migrated north and fans out west of the site. A pond and northerly bend has formed at the mouth of the creek less than about 100 feet west of the site. There is some type of access road through site and onto the beach at that location. An aggregate/concrete batch plant exists east of the site. Morro Bay High School exists north of Atascadero Road. There is significant development east of Highway 1. Highway 1 is 2-lanes in each direction with a full interchange at Atascadero Road.
1972	1" = 341'	Mark Hurd	Mouth of Morro Creek has receded or has been relocated south of the site. Wastewater treatment plant has expanded (Digesters 1 and 2, sedimentation tanks 1 and 2, biofilter 2 shown on 1981 plans), mobile home park is present west of the site, and Atascadero Road is paved and extends along current alignment to the large parking area now located immediately west of the site.. Southern end of the site is still undeveloped sand dunes with trails
1989	1" = 666'	USGS	Similar to 1972 photo and current site condition. Limits of the surrounding land development is similar to today Mobile home parking is present south of the site. Existing sludge ponds andd 1980's plant improvements are present.
1994	1" = 666'	USGS	Similar to 1989 photo.
2002	1" = 666'	USGS	Similar to 1994 photo.
2005	1" = 604'	EDR	Similar to 2002 photo. Mouth of Morro Creek has migrated north on Oceanside of dune, and is immediately west of the existing site.

A wastewater treatment plant is not shown on the historical topographic map from 1951, but is shown on the 1965 map. Based on the review of the historical topographic maps, the plant was likely first constructed between 1951 and 1965. The historical topographic map from 1900 includes a lake that is present just north of the existing plant.

2.4 GENERAL CONDITIONS

This report is preliminary and is not intended for use in the design. Fugro prepared the conclusions, recommendations, and professional opinions of this report in accordance with the generally accepted geotechnical principles and practices at this time and location. This warranty is in lieu of all other warranties, either expressed or implied. This report was prepared for the exclusive use of MWH and their authorized agents only. It is not intended to address issues or conditions pertinent to other parties, projects or for other uses. The report and the drawings contained herein are not intended to act as construction drawings or specifications.

The scope of services did not include any environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere. Any statements, or absence of statements, in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes

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and are not intended to convey engineering judgment regarding potential hazardous/toxic assessment.

Soil and rock deposits can vary in type, strength, and other geotechnical properties between points of observations and exploration. Additionally, groundwater and soil moisture conditions also can vary seasonally or for other reasons. Therefore, we do not and cannot have a complete knowledge of the subsurface conditions underlying the site. The conclusions and recommendations presented in this report are based upon the findings at the points of exploration, and interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed by construction.

3. SITE CONDITIONS

3.1 GEOLOGIC SETTING

The project is located in Morro Bay near the Pacific Ocean coastline, west of hills and mountains that are part of the Coast Ranges geologic and geomorphic province. That province consists of north-northwest-trending sedimentary, volcanic, and igneous rocks extending from the Transverse ranges to the south, into northern California. Rocks of the Coast Ranges province are predominately of Jurassic and Cretaceous age; however, some pre-Jurassic, along with Paleocene-age to Recent rocks are present. The surficial geology in the project vicinity, as mapped by Hall (1979), is shown on Plate 3.

As shown on Plate 3, the site is underlain by Holocene-age alluvial soils deposited primarily by nearby Morro Creek. The alluvial soils consist predominantly of interlayered sands and clays with varying fines and sand content, respectively. Hall maps Holocene-age dune sand deposits near the site along the coastline, and describes them as unconsolidated, white to brown windblown sand of actively moving sand dunes. We observed dune sand deposits bordering Atascadero Road north and west of the site and review of historical aerial photographs shows dune sands covering part of the site in 1937. The dune sand deposits change shape and extent based on storm events and wind and have likely been deposited over and within alluvial soils in the site vicinity.

3.2 FAULTING

The majority of the faults within the Coast Ranges province and the Sierra de Salinas belt generally trend north-northwest. The California Geological Survey (CGS 2002) considers major faulting within the project vicinity to be related to the Los Osos fault, the offshore Hosgri fault, the San Luis Range fault zone (a compilation of several named fault strands), and the Rinconada fault. The CGS fault database consists of active and selected potentially active faults that are considered by the CGS to be capable of affecting regional seismicity in California. Terms used by CGS to describe fault activity are defined below.

Active. Faults that show evidence of displacement during the most recent epoch of geologic time, the Holocene, are classified as active. The Holocene epoch is generally considered to have begun about 11,000 years ago.

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Potentially Active. Faults which displace geologic formations of Pleistocene age but show no evidence of movement in the Holocene period can be considered to be potentially active. Pleistocene time is the period between about two million years ago and 11,000 years ago.

Inactive. Faults which show no evidence of movement during the past two million years and show no potential for movement in the future are classified as inactive.

Fugro utilized the fault search routine in FRISKSP (Blake, 2000) to identify active and potentially active mapped faults and fault segments within a 62-mile radius of the project vicinity. The site coordinates (latitude and longitude) for the site were estimated to be 35.3799° latitude and -120.8604° longitude. Summarized below are eight (8) faults and fault segments that are considered by the CGS (2002) to be the most capable of producing high ground motion within the project vicinity. Additional information is presented in the CGS fault database (CGS, 2002).

Summary of Fault Characteristics

Fault	Fault Type	Approximate Distance From Site (mile)	Maximum Moment Magnitude (M_w)	Fault or Fault Segment Length (km)	Activity
Los Osos (Irish Hills segment)	Reverse	5.4	7.0	44 ± 4	Active
San Simeon – Hosgri	Right-Lateral Strike Slip	7.7	7.5	169 ± 17	Active
San Luis Range (S. Margin)*	Reverse	14	7.2	64 ± 6	Potentially Active
Rinconada	Right-Lateral Strike Slip	15	7.5	190 ± 19	Potentially Active
Casmalia (Orcutt Frontal Fault)	Reverse	33	6.5	29 ± 3	Potentially Active
San Juan	Right-Lateral Strike Slip	36	7.1	68 ± 7	Potentially Active
Lions Head	Reverse	38	6.6	41 ± 4	Potentially Active
San Andreas (Parkfield)	Right-Lateral Strike Slip	41	6.5	36 ± 4	Active

* San Luis Range includes segments of San Luis Bay, Oceano, Wilmar Avenue, Olson, and Santa Maria River faults

The project area is not located within a State of California designated Alquist-Priolo Earthquake Fault Zone (CGS, 1997) and no faults are shown trending through the site on published geologic maps.

The Cambria fault is the closest mapped fault to the project site. Additionally, the Oceanic fault is mapped within the project vicinity. The Cambria fault and Oceanic fault are not

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included in the CGS fault model and consequently are not listed in the above table. However, the San Luis Obispo County Seismic Safety Element (County of San Luis Obispo, 1999) identifies the Cambria fault as potentially active, and there is historic seismicity associated with the Oceanic fault. Therefore, the faults are potential sources of strong ground shaking and ground rupture (or deformation) in the project area. A brief discussion of these faults and the Los Osos fault is provided below.

Cambria Fault. The Cambria fault is mapped approximately 1.3 miles northeast of the site. The northwest-trending fault is mapped locally as a reverse of thrust fault (Jennings, 1994; Kilbourne and Mualchin, 1980).

Oceanic Fault. The Oceanic fault is mapped by Hall (1979) approximately 5.5 miles northeast of the site and is attributed as the source of the 2003 M6.5 San Simeon Earthquake and would therefore be considered active. The northwest-trending fault is interpreted as a segment of the north/northwest-trending West Huasna/Oceanic fault zone. The West Huasna/Oceanic fault zone has been interpreted by Hall (1973) as a series of low- to high-angle reverse faults cut by a younger set of nearly vertical faults.

Los Osos Fault. The Los Osos fault zone is mapped the base of the hills south of the community of Los Osos (CGS, 2002; PG&E, 1988; Lettis & Hall, 1994; Asquith, 1997). Lettis & Hall (1994) describe the Los Osos fault zone as a series of discontinuous, subparallel and en echelon fault traces that extend from the offshore Hosgri fault zone to Lopez Reservoir, a distance of about 35 miles. Portions of the fault east of Los Osos near the City of San Luis Obispo have been zoned active and are designated as an Alquist-Priolo earthquake fault hazard zone by CGS.

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3.3 SUBSURFACE CONDITIONS

The soil and groundwater conditions were characterized for the preliminary study based on the results of previous field explorations performed by ESP and GTC. The locations of the previous explorations are shown on Plate 2. Logs of our interpreted CPT soundings performed by ESP are presented in Appendix A. Correlations developed by Robertson and Campanella (1988) were use to classify soils encountered in the CPT soundings.

The site is underlain primarily by alluvium and dune sand deposits. The site for the existing plant appears to have been cut into the sand dunes that previously occupied the site. The alluvium and dune sand deposits were locally overlain by approximately 1 to 4 feet of artificial fill. The fill consisted mainly of medium dense to dense sand with varying fines and gravel content. The artificial fill is likely associated with site preparation and grading for the existing plant.

Dune Sand Deposits. Dune sand deposits were encountered beneath the artificial fill and at the existing ground surface across the site. ESP encountered dune sand deposits to depths ranging from approximately 10 to 25 below the existing ground surface that were interlayered with discontinuous 2-to-5-foot thick layers of soft to stiff silt and clay alluvium. GTC encountered approximately 5 to 10 feet of dune sand deposits beneath the artificial fill that was

underlain by alluvium to the maximum depth explored in their borings, approximately 40 feet below the existing ground surface. The dune sands consisted of predominantly medium dense to dense sand with varying fines content. Layers (less than 2 feet thick) of loose sand, silt and clay (possibly interbedded alluvium) encountered at various depths within the CPT soundings between approximately 10 and 17 feet below the existing ground surface.

Alluvium. Alluvium was encountered beneath the dune sand deposits to the maximum depth explored, approximately 55 feet below the existing ground surface in CPT-2. The alluvium consisted mainly of interlayered firm to stiff clay and silt and medium dense to dense sand. Layer of firm to stiff clay and silt are interbedded within the alluvium. An approximate 6-foot thick layer of medium dense gravel with sand and clay was encountered in ESP boring B-2. GTC and ESP encountered stiff to very stiff clay in selected CPT soundings and below a depth of approximately 25 to 30 feet.

3.4 GROUNDWATER

Groundwater was encountered at a depth of approximately 10 feet below the ground surface in borings performed by ESP in October 2009. Groundwater was encountered at depths between approximately 7 and 13 feet below the ground surface in borings performed by GTC in August 1980. We assumed a groundwater depth of 10 feet below the existing ground surface in our evaluation of the site conditions. Variations in surface and groundwater conditions will likely occur as a result of water t level fluctuations and migration of the mouth of Morro Creek, possible tidal and ocean front influences, changes in precipitation, irrigation, runoff, and other factors.

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4. GEOLOGIC HAZARDS

The following geologic hazard assessment is based on review of previous studies performed at the site, published geologic maps, historical aerial photographs, and observations made during our site visits. Our assessment was performed to evaluate the potential for the project site to be impacted by geologic hazards.

4.1 SEISMICITY AND STRONG GROUND MOTION

4.1.1 Historic Seismicity

The site is located within a seismically active region of Central California. A summary of Magnitude 2 and greater seismic events recorded from 1933 through March 2008 by the Advanced National Seismic System (ANSS, 2008) and Clark et al. (1994) are presented on Plate 6 - Historical Seismicity Map. Examples of relatively strong ground motion that has reportedly been experienced near the project area are the seismic events of 1830, 1857, 1913, 1916, 1917, 1952, 1966, 1980, and 2003.

The 1830 event is estimated to be an approximately M5 earthquake that occurred from a poorly located source near San Luis Obispo. The effects of the 1830 event were generally observed between the Los Osos and Rinconada faults. The 1857 event (the Fort Tejon

earthquake) occurred on the Mojave segment of the San Andreas fault, and reportedly resulted in damage in central and southern California. The 1913 event is estimated to be an approximately M5 earthquake that occurred along the southwestern margin of the San Luis/Pismo block near Arroyo Grande. The 1916 event is estimated to be an approximately M5 earthquake that occurred near Avila, possibly along the Los Osos fault or faults along the southwestern margin of the San Luis/Pismo block. The 1917 event is estimated to be an approximately M5 earthquake that occurred near Lopez Canyon between the Rinconada and West Huasna faults. The 1952 earthquake is estimated to be a M6 earthquake occurring within the Nacimiento Fault Zone. The 1966 event (the Parkfield earthquake) is estimated to be an approximately M6.0 earthquake that occurred on the San Andreas fault. The 1980 event is estimated to be an approximately M5 earthquake that occurred offshore near Point Sal along the Casmalia fault zone, and near its intersection with the Hosgri fault.

The epicenter of the 2003 San Simeon earthquake was located approximately 26 miles northwest of the project site. A ShakeMap for the site developed by the California Integrated Seismic Network (CISN 2003) shows that the project site likely experienced moderate ground motion during the M6.5 earthquake. The peak horizontal ground acceleration at the project site, as estimated from the ShakeMap, was likely about 0.15g.

4.1.2 Ground Motion Study

The ASCE (2005) design code and the California Building Code (CBC 2007) require structures to be designed for earthquake effects that are two-thirds ($^2/_3$) of the Maximum Considered Earthquake (MCE) effects. The MCE is defined by comparing a probabilistic MCE, deterministic MCE and some lower boundaries.

Based on subsurface information presented by Earth Systems Pacific (ESP 2009) and GTC (1980), a soil profile was assumed for the attenuation relationships applied in the seismic hazard analyses. We approximated an average site shear wave velocity of 890 feet per second (270 meters per second) for the upper 100 feet (30 meters) of subsurface material. We performed a ground motion study in accordance with ASCE (2005) and the 2007 CBC. The estimated seismic conditions for the MCE were a magnitude 7.34 earthquake having a peak ground acceleration of approximately 0.34g at the site. The estimated MCE values are similar to those estimated by ESP: a 0.33g peak ground acceleration and magnitude 7.56 earthquake. The MCE is stronger than the estimated seismicity from the 2003 San Simeon Earthquake having an estimated 0.15g peak ground acceleration (approximately ½ the MCE) for the magnitude 6.5 event. Plotted acceleration response spectra will be provided in the design-level report for the project.

4.2 SURFACE FAULT RUPTURE

The site does not lie within an Alquist-Priolo fault rupture hazard zone. Based on our data review, no known active or potentially active faults trend toward or traverse the project site. Therefore, the potential for ground rupture to occur at the site appears to be low.

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4.3 LIQUEFACTION

Liquefaction is the loss of soil strength due to an increase in soil pore water pressures that results from seismic ground shaking. Liquefaction typically occurs within relatively loose to medium dense sandy soil that is below the groundwater table. If those conditions are present and strong ground motion occurs, portions of the soil column could liquefy, depending upon the intensity and duration of the strong ground motion. Liquefaction analyses were performed for the MCE (M7.34, 0.34g) using CPT data from ESP (2009) and an assumed groundwater depth of 10 feet below the existing ground surface. A second analysis was performed to evaluate the potential for liquefaction using the estimated seismic data for the San Simeon Earthquake and to compare those results to the MCE.

The results of our liquefaction analyses for soils encountered in the CPT soundings is summarized graphically on Plate 6. The red line on the plate is the estimated CPT tip resistance that is needed to resist liquefaction for the seismic conditions considered. A blue zone between the red line and the CPT tip resistance indicates a zone of potentially liquefiable soil.

For the MCE, zones of soil were encountered in the CPT soundings that are considered potentially liquefiable. Potentially liquefiable soil conditions were generally within approximately 1- to 4-foot thick layers of loose to medium dense sand. The potentially liquefiable sand layers appear at two general locations in CPT-1 through CPT-3; near the groundwater level between approximately el. 10 to 3 feet and within the interbedded sand, silt, and clay between approximate el. -20 to -33. CPT-4 also encountered potentially liquefiable layers between approximate el. -2 to -10 feet. The potentially liquefiable soil layers were encountered as shallow as the depth to groundwater, 10 feet below the existing ground surface.

Plate 7 summarizes liquefaction analyses based on the estimated ground motion for the San Simeon Earthquake. The analysis was performed using a M6.5 for the San Simeon Earthquake with a corresponding peak ground acceleration of 0.15g. The results of the liquefaction analysis for the San Simeon Earthquake is summarized graphically on Plate 7. The results suggest the soil encountered in the CPT soundings would have a low potential for liquefaction using the estimated peak ground acceleration for the San Simeon Earthquake. The penetration resistance in each of the soundings exceeded the minimum CPT resistance that would be needed to cause liquefaction.

The results of the analyses are consistent with our discussions with plant personnel (Mr. Bruce Keogh, Mr. Les Girvin) whose accounts of the San Simeon Earthquake did not include evidence of liquefaction or seismic settlement. Plant personnel reported vertical cracks in the masonry walls of the electrical room and around a door in the administration building, and some separation near the IPS (influent pump station).

4.4 SEISMIC SETTLEMENT

Seismic settlement can occur in loose to medium dense, saturated or unsaturated soils as a result of consolidation or shakedown of the soil matrix in response to seismic shaking.

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Loose to medium dense sand encountered within the dune sand deposits and alluvium is prone to seismic settlements in response to the estimated ground motion.

Seismic settlement analyses were performed in association with the liquefaction evaluation for the MCE summarized on Plate 6. The black line shows the estimated cumulative seismic settlement between the bottom of the sounding and the ground surface considering all soil layers. We estimate that the cumulative seismic settlement due to liquefaction is approximately 1 ½ to 3 ½ inches for the MCE, the majority of which is estimated to occur above el. -10 feet (within about 15 to 30 feet of the ground surface). However, liquefaction of soil layers within the lower portions of the soil column could limit seismic settlement. We estimate that an additional small amount (less than about ½-inch) of seismic settlement could occur in the sand layers above the groundwater table, for a total of up to about 4 inches of estimated settlement for the MCE. The estimated seismic settlement is expected to occur differentially over the site as a result of the complex and heterogeneous composition of the alluvium. As shown on Plate 7, of the estimated seismic settlement for the San Simeon Earthquake ground motion is negligible (less than about 1/8-inch).

4.5 LATERAL SPREADING

Lateral spreads can occur in loose, soft, or moderately dense and weakly cemented soils, or in association with liquefaction. We estimate there is potential for approximately 4 feet of lateral spread displacement to occur at the site based on procedures by (Youd, et. Al 2002), using the existing subsurface data, the results of our ground motion study, and if there were continuous liquefiable soil layers at the site that extended to the beach front (which appears unlikely). We therefore conclude for the preliminary evaluation that the potential for lateral spreading is uncertain, and should be further evaluated during the design-level to evaluate if there are sufficient, continuous liquefiable layers to present a significant hazard to the plant.

The preliminary analyses assumed continuous soil layers, and a gently sloping ground condition of approximately 0.8% based on elevations in Wallace Group (2009) report. We used the design earthquake moment magnitude of 7.34 and the modal R-distance of 8.5 miles (13.7 kilometers) from the probabilistic seismic hazard analysis using the USGS 2008 Interactive Disaggregation (beta) website. Average fine content and mean grain size were estimated from the results of laboratory testing by GTC and ESP for a thickness of liquefiable soil with $N_{1(60)}$ less than 15 blows per foot of approximately 3 feet (1 meter).

4.6 SUBSIDENCE AND COLLAPSE

The site is located near the coast with shallow groundwater and is not in an area where the withdrawal of subsurface fluids is known to have caused ground subsidence. The soils previously encountered at site generally have low compressibility, and should not be susceptible to significant compression due to lowering of local water levels. The evaluation of the potential for subsidence or settlement, such as due to localized dewatering of soil layers, should be evaluated further during the design-level field exploration.

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4.7 LANDSLIDING/SLOPE INSTABILITY

The project site is on relatively flat terrain away from slopes in a developed area of Morro Bay. It is therefore our opinion that there is a low potential for subsidence to impact the potential project site.

4.8 EROSION

Graded cut and fill slopes associated within the site development will be subject to sheet and rill erosion. Erosion of soils can be accelerated where soils are exposed directly to runoff and/or areas of concentrated storm runoff, such as at culvert outlets. Site drainage and landscape improvements can be designed to reduce the potential for soil erosion.

4.9 EXPANSIVE SOILS

Soil conditions near the ground surface generally consist of sand that have a low potential for expansion. No special mitigation to address expansive soils is anticipated. Naturally OCCURRING asbestos

Naturally occurring asbestos (NOA) is common in serpentine rock throughout San Luis Obispo County. The California Air Resources Board has identified serpentine rock as having the potential to contain asbestos. The dune sand and alluvium that underlie the site would not contain minerals classified as having a significant source of NOA. Therefore, it is our opinion that there is a low potential for NOA to impact the project.

4.10 FLOODING AND INUNDATION

A flood hazard analysis was performed for the site by the Wallace Group (2009). The Wallace Group identified that the site is within a designated FEMA flood zone and estimate flood elevations and flows associated with Morro Creek. We understand that the MWH design for the plant will include raising site grades and structures to mitigate flooding hazards for the project.

Inundation can occur when a large body of water is released typically in response to a failure or breach in a dam. The San Luis Obispo County Office of Emergency Services has a web site with inundation maps for dams and levees within the San Luis Obispo County[1]. The site is not downstream of large dams that could inundate the site in response to breach or failure. The nearest jurisdictional dam to the site is the Whale Rock Dam near Cayucos, located about 3 miles north of the site. The inundation zone for Whale Rock Dam does not extend near the site.

During reconnaissance of the watershed above site performed using Google Earth (www.google.com), we observed approximately 23 acres of agricultural ponds on a hill top approximately 5 miles upstream of the site. The ponds are located above the avocado orchards on the north side of Highway 41. The ponds are likely not jurisdictional, as determined by the

[1] http://www.slocounty.ca.gov/OES/plans/Dam_and_Levee_Plan_Maps.htm

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California Division of Safety of Dams, because there is no record of an inundation map for the reservoirs which would be required for a jurisdictional dam. Given the location and size of the reservoirs it unlikely the reservoirs present a significant inundation hazard to the site, however we do not have any other information about the reservoirs, their design or construction.

4.11 TSUNAMIS

Tsunamis are ocean waves generated in response to earthquakes. Tsunamis waves can be generated by near or far field earthquakes, and a series of waves from a single earthquake can travel across the ocean in response to an earthquake carrying with them massive amounts of water. The Morro Bay-Cayucos Wastewater Treatment Plant is located approximately 20 feet above sea level and on coastal dunes that front the Pacific Ocean and Morro Strand State Beach. The Tribune news paper[2] reported that tsunami run-ups of several feet were recently observed in Morro Bay harbor in response to the February 2010 Chilean M8.8 Earthquake.

The State of California (2009) has created tsunami inundations maps for emergency planning, and shows the site as being within a designated tsunami inundation area. The map designates a potential runup to approximately elevation 34 feet in the site vicinity. The site is within the inundation zone indicated on the map. The map notes that the designated zone is not a complete inundation of the entire coastline below el. 34 feet, but designates areas that could be susceptible to or could need to be evacuated in response to a tsunami. The County and City of Morro Bay do participate in the tsunamis warning system described on the County web site.

5. CONCLUSIONS AND RECOMMENDATIONS

We prepared the conclusions and recommendations for this report based on our preliminary geotechnical evaluation of the site conditions and a review of available geotechnical information for the existing plant. The conclusions are preliminary and subject to change based on information that will be obtained during the design-level geotechnical evaluation.

5.1 SUMMARY OF FINDINGS

- Based on existing data, the site is underlain primarily by dune sand deposits and alluvium consisting mainly of medium dense to dense sand and soft to stiff clay and silt. Previous grading performed as part of the site's development has resulted in flattening of the dunal terrain that previously occupied the site, filling of low areas, and cuts in to the dune sand. Groundwater was encountered at depths ranging between approximately 7 and 13 feet below the existing ground surface in previous investigations.

- The site is located within a seismically active region of Central California that has been subjected to strong ground motions in response to historical earthquakes that have occurred in relatively close proximity to the site, such as the M6.5 San Simeon Earthquake in 2003. There was limited structural damage to the plant in response to

[2] http://www.sanluisobispo.com/2010/02/27/1047602/chile-earthquake-causes-surge.html

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the 2003 earthquake; however, we know of no reports to suggest there was evidence of liquefaction or seismic settlement in response to the earthquake. Our analyses suggest that the ground motions from the San Simeon Earthquake were insufficient to trigger liquefaction at the site and were also less than the ground motions that will be used for design. Plant personnel reported vertical cracks in the masonry walls of the electrical room and around a door in the administration building, and some separation near the IPS (influent pump station). The design of the facility should consider the potential for the site to be subject to strong ground motion in response to nearby or regional earthquakes.

- The design earthquake (MCE) is a M7.34 event having a corresponding peak ground acceleration of about 0.34g. Based on the conditions encountered, there is a potential for liquefaction and seismic settlement to impact the design of the proposed improvements. The potentially liquefiable soils were mainly encountered in the sandy soils above el. -10 feet (within the upper 15 to 30 feet of the site) and in thin (less than 1 foot) isolated sand layers encountered below clayey alluvium at depths of about 40 to 45 feet. The liquefaction hazards appear to vary significantly over the site, and consideration and mitigation for the hazard could vary during design.

- The excavation for selected below grade structures may extend below potentially liquefiable soils or help limit the impact of seismic settlement that could occur in association with liquefaction. If design of the proposed improvements cannot withstand the estimated seismic settlements, subsurface stabilization such as deep foundations, vibrocompaction with stone columns, excavation and replacement, or other methods could be used to help mitigate the hazard.

- The site is located within designated flood and tsunami inundation zones. Development should consider the proximity of the site to the Pacific Ocean and nearby Morro Creek and the potential for flood and tsunami inundation.

- We understand that up to 7 feet of new fill will be placed to raise the site above estimated flood levels. We preliminary estimate (using presumptive values) that placement of the fill could cause approximately 3 to 9 inches of settlement due to compression of relatively soft to firm clay layers encountered in the CPT soundings, and that the settlement could take several months to occur. A settlement waiting period may be needed to allow for the settlement to occur prior to construction of the improvements. The design and construction of the proposed improvements should consider settlement of the underlying soils as a result of placement of the new fill.

- If design of the proposed improvements can withstand the estimated seismic settlement or subsurface stabilization is performed in advance of their construction, we expect that typical 1- to 2-story structures, below grade structures, and moderately loaded structures could be supported on shallow foundations supported on compacted fill. Deep foundations or ground improvement can be used to help limit settlement, and should be evaluated as part of the design-level geotechnical study. Relatively heavy structures such as tall buildings or large tanks will need to consider the potential for total and differential settlement of the underlying fine

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grained layers encountered at the site and may require special attention relativel to settlement and seismic hazards during design.

5.2 SEISMIC CONSIDERATIONS

5.2.1 Preliminary Seismic Data

Structures should be designed to resist the lateral forces generated by earthquake shaking in accordance with the building code and local design practice. This section presents seismic design parameters for use with the 2007 California Building Code (CBC). The CBC is based on the 2006 International Building Code (IBC). The site coordinates and USGS interactive web page "Seismic Design Values for Buildings" (USGS 2008) was used to obtain seismic design criteria. Based on these criteria, the preliminary seismic data for use with code-based designs are:

Preliminary Seismic Data

California Building Code	Seismic Parameter	Value
Site Coordinates	Latitude, degrees	35.3799
	Longitude, degrees	-120.8604
Section 1613.5.1 Figure 1613.5	Ss , Seismic Factor, Site Class B at 0.2 sec	1.246
	S_1, Seismic Factor, Site Class B at 1 sec	0.482
	Site Class	S_D, Stiff soil
Section 1613.5.3 Table 1613.5.3(1)	Fa, Site Coefficient for Site Class D	1.002
Section 1613.5.3 Table 1613.5.3(2)	Fv, Site Coefficient for Site Class D	1.518
Section 1614A	S_{MS}, Site Specific Response Parameter for Site Class D at 0.2 sec.	1.126
	S_{M1}, Site Specific Response Parameter for Site Class D at 1 sec.	0.630
	S_{DS} = 2/3 S_{MS}	0.751 (S_{DS} value from project design response spectrum in design level report should be used for design)
	S_{D1} = 2/3 S_{M1}	0.420 (S_{D1} value from project design response spectrum in design level report should be used for design)

Based on the seismic design parameters calculated by the USGS interactive web page "Seismic Design Values for Buildings" (USGS, 2008), and per 2007 CBC Section 1613.5.6, structures of Occupancy Category I, II, III, and IV (defined in 2007 CBC Table 1604.5) should be designed according to Seismic Design Category "D". Plotted acceleration response spectra will be provided in the design-level report for the project.

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5.3 LIQUEFACTION AND SEISMIC SETTLEMENT

Subsoil stabilization using vibro-replacement with stone columns was performed at the site for improvements to the plant in the 1980's. The stabilization was specified to densify and decrease the liquefaction potential of soils under the site. Plate 2 shows the approximate limits of subsoil stabilization performed in specific areas of the existing plant, north of the currently proposed improvements. Because the San Simeon Earthquake appears to have generated ground motions insufficient to cause liquefaction at the site, the historic data is not representative of the liquefaction hazards that could impact the site for the design earthquake event or evaluate the benefits of the past subsoil stabilization.

As discussed in Sections 4.3 and 4.4, zones of potentially liquefiable soil were encountered in the CPT soundings that vary in continuity and aerial extent. The preliminary estimates of seismic settlement for soil conditions at the site range from small amounts of settlement to up to approximately 4 inches without grading. Because of the heterogeneous nature of the soils encountered, the estimated total settlement is likely to occur differentially over the site and some structures. Below grade structures may locally extend below potentially liquefiable soil layers or help to reduce the estimate settlement; however, further evaluation of that scenario should be evaluated during the design-level geotechnical study.

Some building structures can typically be designed with stiffened slabs or grade beams to help tolerate differential and seismic settlement. However, if the design of the proposed improvements cannot withstand the estimated seismic settlement, structures may need to be supported on deep foundations, excavation and replacement, or ground modification methods such as vibro-compaction with stone columns (such as the subsoil stabilization previously performed) could be performed in advance of construction of structures to reduce the potential seismic settlement and liquefaction. Because of the heterogeneous nature of the site, the foundation support, seismic settlement hazard, and mitigation will likely need to be performed on a site-by-site, structure-by-structure basis during design.

5.4 SETTLEMENT FROM NEW FILL CONSIDERATIONS

We understand that up to approximately 7 feet of new fill will be placed in areas of the site to raise it out of the flood plain. Placement of the new fill should consider the potential for settlement due primarily to consolidation of the soft to firm silt and clay layers encountered beneath the site. We preliminarily estimate that about 3 to 9 inches of post-construction settlement could occur at various locations over the site due to placement of the fill, and that the settlement could take several months or more to occur. The use of wick drains to accelerate the settlement, settlement waiting periods, and better characterization of the subsurface conditions should be addressed in the design-level report. The settlement will likely need to be allowed to occur prior to construction of the improvements to reduce the potential for differential settlement of the structures and the need for deep foundations.

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5.5 SITE PREPARATION AND GRADING CONSIDERATIONS

5.5.1 Clearing and Grubbing

Clearing and grubbing should be provided to prepare the site for grading. Demolition areas within the proposed limits of above grade and below grade structures should be cleared of old foundations, existing pavements, and soils disturbed by the construction. Depressions or disturbed areas left from clearing and grubbing should be replaced with compacted fill.

5.5.2 Above Grade Structures and Buildings

The near surface soils encountered generally consisted of loose to medium dense sand. Site preparation and grading will be needed to provide relatively uniform support for at-grade structures, buildings supported on shallow foundations, equipment slabs or other improvements. Grading would likely consist of removing the upper approximately 3 to 5 feet of the existing soils and replacing that material with compacted fill. The excavation should typically extend at least 5 feet outside the proposed building or structure footprint. Prior to placing the compacted fill, the bottom of the excavation should be graded such that a relatively uniform thickness of fill will be provided below the foundations, and be compacted to prepare for subsequent fill placement.

5.5.3 Below Grade Structures

We expect that excavations for below grade structures will extend to approximately 10 to 20 feet below the existing ground surface. Based on the results of our field investigation, we expect that the deep excavations below approximately 10 feet will likely encounter groundwater and/or wet subgrade conditions. For preliminary design, we recommend the design consider that approximately 1 to 2 feet of open graded gravel or permeable material, wrapped in geotextile, will be provided over the bottom of the excavation to stabilize the subgrade and provide a suitable working mat for construction of below grade structures. Dewatering and shoring systems will likely need to be provided in association with the construction of below grade structures as discussed in this report.

5.6 FOUNDATION DESIGN CONSIDERATIONS

Site conditions relating to seismic settlement and consolidation of the soft ground due to fill placement should be addressed prior to designing structure foundations as previously discussed in this report. If liquefaction and post-construction settlement is mitigated in advance of foundation construction, below grade and lightly loaded structures can likely be supported on shallow foundations.

We expect that typical 1-to 2-story and moderately loaded structures (wall loads less than about 4 kips per foot, column loads less than 50 kips) can likely be supported on spread footing foundations bearing in compacted fill materials. Larger structures, if proposed, should be evaluated during the design-level and may need to consider the use of deep foundations to limit settlement. Below grade structures (sedimentation basins and oxidation ditches) typically impose less load on the foundation support soil and can likely be supported on a thickened mat

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type foundation or slab. If those structures are to be constructed above existing site grades, they will likely need to be support on deep foundation or improved ground. For preliminary design, spread footing foundations founded in compacted fill or mat foundation for below grade structures can be designed using a maximum allowable bearing pressure of 2,000 to 3,000 pounds per square foot. Continuous footings should be designed with a width of at least 1 foot. Isolated pad footings should be at least 1.5 to 2 feet square. Spread footing foundations should be embedded at least 1.5 feet into compacted fill material, and at least 2 feet below the lowest adjacent exterior grade or finished slab elevation whichever is deeper.

The recommended maximum allowable bearing pressure can typically be increased for each additional foot of footing width or embedment exceeding the recommended minimums depending on the depth to groundwater at the footing location. The toe pressure for retaining walls or eccentrically loaded footings can exceed the recommended maximum allowable bearing pressure provided the resultant static force acts within the middle third of the footing.

5.6.1 Settlement Considerations

Structures will result in additional settlement of the foundation support soil in addition to settlement associated with seismic hazards and fill placement that were previously discussed in this report. For structures whose foundations are placed near existing grade on compacted fill material, settlement of the foundation soil is likely to occur in response to elastic loading of the soils and as a result of compression of the compacted fill material itself. We expect that total and differential settlements for footings bearing in compacted fill will generally be less than 1-inch for the maximum allowable bearing pressures described above and one to two-story above grade structures.

Relatively large or heavily loaded above and below grade structures will need to consider total and differential settlements of the underlying soft to stiff fine grained layers encountered within the sand. If soft soil is present near the base of below grade structures, the foundation soil could be prone to greater amounts of settlement. Settlement of below grade structures should be further evaluated in our design level geotechnical report based on the anticipated foundation types and depths.

5.7 GRADED SLOPES

Graded cut and fill slopes likely can be designed to a slope inclination of 2h:1v or flatter. Retaining structures or reinforced slopes can be provided to allow for steeper slopes, if needed.

5.8 DRAINAGE AND EROSION CONSIDERATIONS

Drainage should be provided such that surface water does not run over slopes or pond on pavements, slabs, or adjacent to foundations. Downspouts should be provided to collect roof drainage and direct the water to drainage pipes or areas away from the building. Landscaping and maintenance of slopes should be provided to assist vegetation to be established on slopes, and reduce the potential for erosion. The top of slopes should be graded to direct drainage away from the slopes, or be provided with dikes and ditches that will direct surface water to

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controlled drainage structures. Concentrated flows and runoff should not be permitted to discharge onto slopes. Down drains, solid pipes, or lined ditches should be provided to carry water to the base of slopes. Energy dissipation and erosion control devises should be provided at the outlet of drainage pipes and in areas of concentrated flow and runoff to reduce the potential for erosion.

5.9 CONSTRUCTION CONSIDERATIONS

5.9.1 Excavation

The dune sand deposits and alluvium encountered beneath the project site consist of medium dense to dense sand and firm to stiff fine-grained materials. These types of soils likely can be excavated with typical heavy-duty construction equipment in good working order.

5.9.2 Use of On-site Soil

On-site soil consisting of sandy-type material that is free of organics, oversized rocks, pavements, and other deleterious materials should be suitable for use as compacted fill in building and pavement areas. Sandy materials should also be suitable for placement as backfill around below grade structures. The clayey on-site soils should not be considered suitable for use as structure backfill, or fill to be placed below building and pavement areas. The on-site soils likely are not suitable for select material such as backfill for pipelines and base for roadways. On-site soil excavated from below the groundwater table will likely not be suitable for compacted fill as excavated. Drying or treating the soils may help to reduce the water content of the excavated material and make the material suitable for compaction.

5.9.3 Dewatering

We expect that excavations extending approximately 5 to 10 feet or more below existing site grades (depending on location) could encounter groundwater at the time of construction. The depth to groundwater can fluctuate at the site due to its proximity to the ocean and nearby Morro Creek. The contractor should be responsible for both designing and maintaining the dewatering system for construction. Gravel will likely need to be placed over the base of excavations to help stabilize the wet subgrade and provide a working mat for construction. Dewatering facilities, such as sump pits, wells, and well points should be designed with filters such that sand and fine-grained materials are not removed from the soil during dewatering operations. Dewatering facilities should be installed prior to beginning excavation, and time should be allowed for lowering of the groundwater table before beginning excavation. Shoring systems, such as continuous sheet piling, should be embedded adequately below the base of the excavation to cutoff groundwater and help stabilize the base of the excavation.

5.7.2 Temporary Slopes and Shoring

The soil encountered in previous investigations consists of loose to medium dense sand and soft to firm clay and silt and should not be considered capable of maintaining a stable vertical slope. Temporary slopes should be braced or sloped according to the requirements of

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OSHA. Based on review of OSHA guidelines and the soil conditions encountered within the anticipated depths of excavation, temporary slopes without shoring should be excavated at inclinations of 1.5h:1v or flatter. Vertical shoring consisting of sheet piles or similar construction systems can be provided in lieu of flattening slopes, if needed, to limit the size of the excavation or to protect adjacent structures.

Where excavations are in close proximity to existing structures, shoring systems should be designed with consideration of the adjacent foundation and structural loads. The shoring should be designed for the additional load and have provisions to reduce the potential for settlement of adjacent structures, pipelines, or roadways. Generally, surcharge loads from existing structures can be neglected if the structure is behind a 1:1 line projected upwards from the nearest bottom edge of a shored trench excavation, or the building is setback at least 10 feet horizontally from properly sloped excavations. If excavations are made within this zone of influence, the contractor should design the slope or shoring system for the additional surcharge load.

6. CONTINUATION OF SERVICES

The geotechnical considerations in this report are intended for preliminary planning and estimating costs associated with developing the site. The recommendations are preliminary based on existing subsurface information. A more comprehensive design-level geotechnical investigation will be completed in support of the final design of the proposed improvements.

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End of Text

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PLATES

BASE MAP SOURCE: USGS 7.5' Morro Bay North 1995 Quadrangle.

NORTH

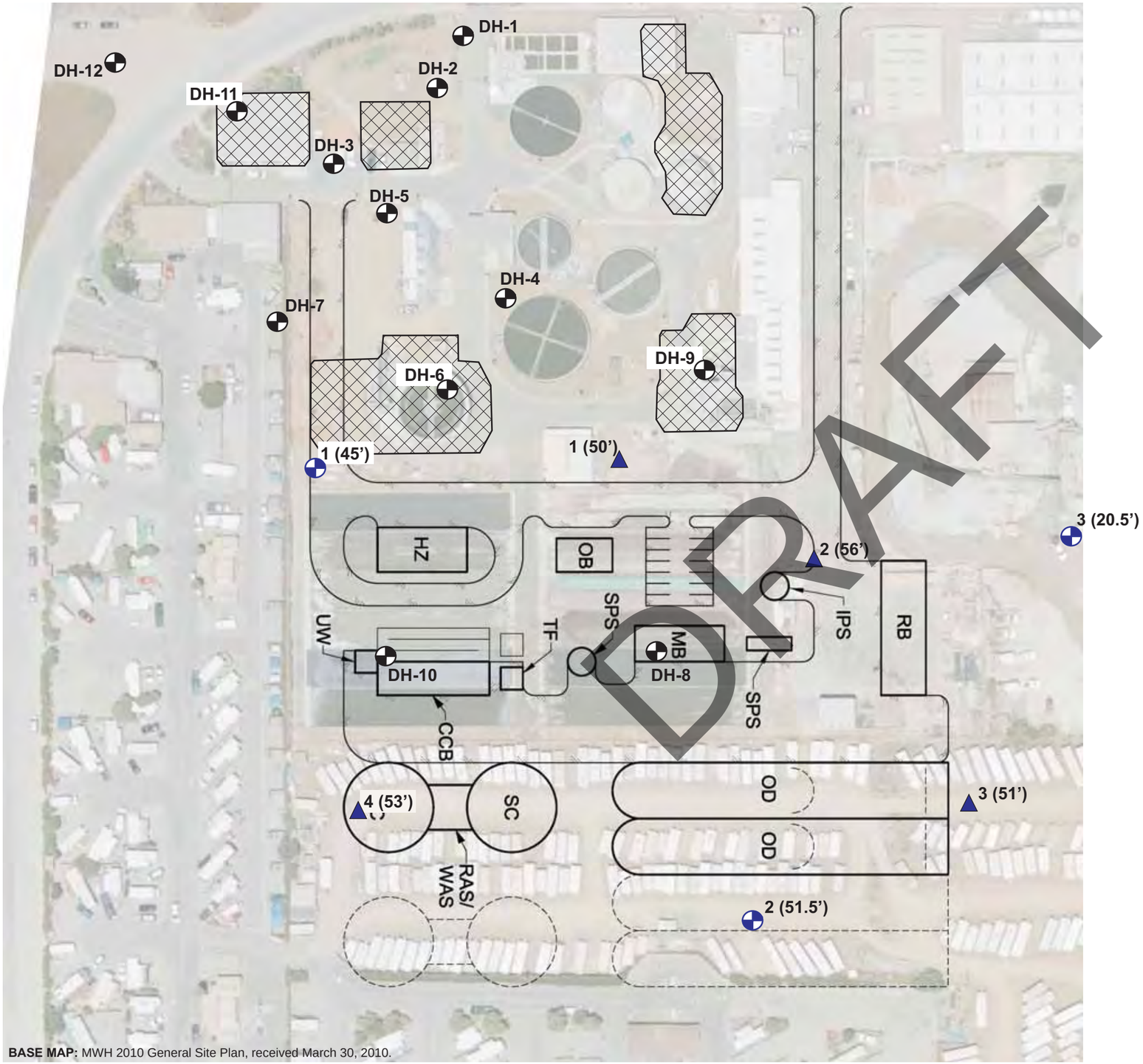
0 1000 2000

FEET

VICINITY MAP

Morro Bay- Cayucos WWTP Upgrade Project
160 Atascadero Road
Morro Bay, California

PLATE 1



BASE MAP: MWH 2010 General Site Plan, received March 30, 2010.

NORTH

0 50 100

FEET

1" = 100'

LEGEND

1 (20') Earth Systems Pacific hollow-stem auger boring - October 2009

1 (20') Earth Systems Pacific CPT sounding - October 2009

DH-1 Geotechnical Consultants, Inc. boring - September 1980

Limits of 1983 subsoil stabilization

CCB CHLORINE CONTACT BASIN
HZ HOUSEHOLD HAZARDOUS WASTE
IPS INFLUENT PUMP STATION
MB MAINTENANCE BUILDING
OB OPERATIONS BUILDING
RAS RETURN ACTIVATED SLUDGE
RB RESIDUALS BUILDING
SC SECONDARY CLARIFIERS
SP STANDBY POWER
SPS SECONDARY PUMP STATION
TF TERTIARY FILTER
UW UTILITY WATER
WAS WASTE ACTIVATED SLUDGE

All locations and dimensions are approximate.

Dimensions and layout may be distorted relative to overlay on aerial photography.

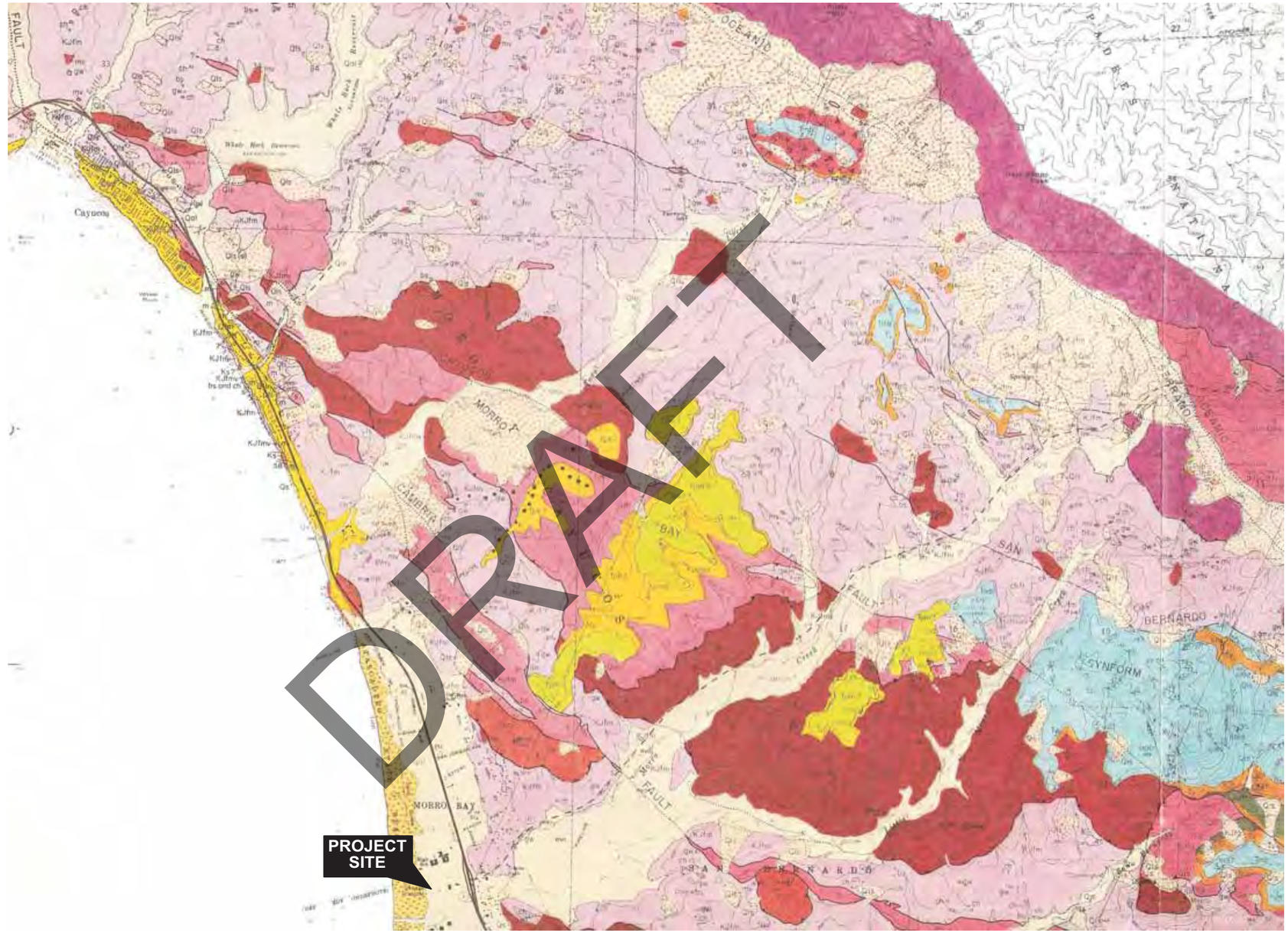
Plant layout is preliminary and subject to change.

SITE PLAN
Morro Bay - Cayucos WWTP Upgrade Project
160 Atascadero Road
Morro Bay, California

PLATE 2

MAP UNITS

Qal	Alluvial deposits
Qs	Dune sand deposits
Qls	Landslide deposits
Qpr	Paso Robles Formation
Tpm	Pismo Formation, Miguelito member
Tpe	Pismo Formation, Edna member
Tpes	Pismo Formation, Edna member, siltstone
Top	Obispo Formation, perlitic glass and breccia
Tr	Rincon Shale
Td	Dacite, porphyritic-aphanitic dacite
Tdf	Dacite, fine-grained dacite or flow facies
KJfg	Franciscan Rocks, Graywacke and micrograywacke
KJfmv	Franciscan Rocks, Metavolcanics
KJfm	Franciscan Rocks, Mélange
ch	white, red, or green chert
mv	metavolcanic
gw	graywacke
S	Serpentinite



NORTH

0 2000 4000

FEET

M:\Drafting\JOBFILES\2007\3014.026\3014.026geo.cdr

BASE MAP SOURCE: Geologic Map of the the San Luis Obispo-San Simeon Region, USGS Misc. Investigations Series Map I-1097, Sheet 3 of 3 (Hall, et al., 1979).

LEGEND

Contact - Dashed where approximately located or inferred; queried where doubtful; dotted where concealed

High-angle fault - Dashed where approximately located or inferred; dotted where concealed and inferred; queried where uncertain. Arrows show relative direction of movement on cross sections when known; queried where uncertain.

Thrust or reverse fault - Dashed where approximately located or inferred, dotted where concealed and inferred; queried where concealed or doubtful. Sawteeth on upper plate. Dip of fault plane between 30° and 80°

Photo lineament - Queried where uncertain

Synform - Trace of axis at surface. Dashed where approximately located. Flanks coverate downward in folds and in rocks whose stratigraphic sequence is unknown.

Antiform - Trace of axis at surface. Dashed where approximately located. Flanks diverge downward in folds and in rocks whose stratigraphic sequence is unknown.

Strike and dip of beds uncertain

Marker beds

Conglomerate or gravel bed

Sandstone

Siltstone or diatomaceous

siltstone

Tuff

30 Strike and dip of flow banding

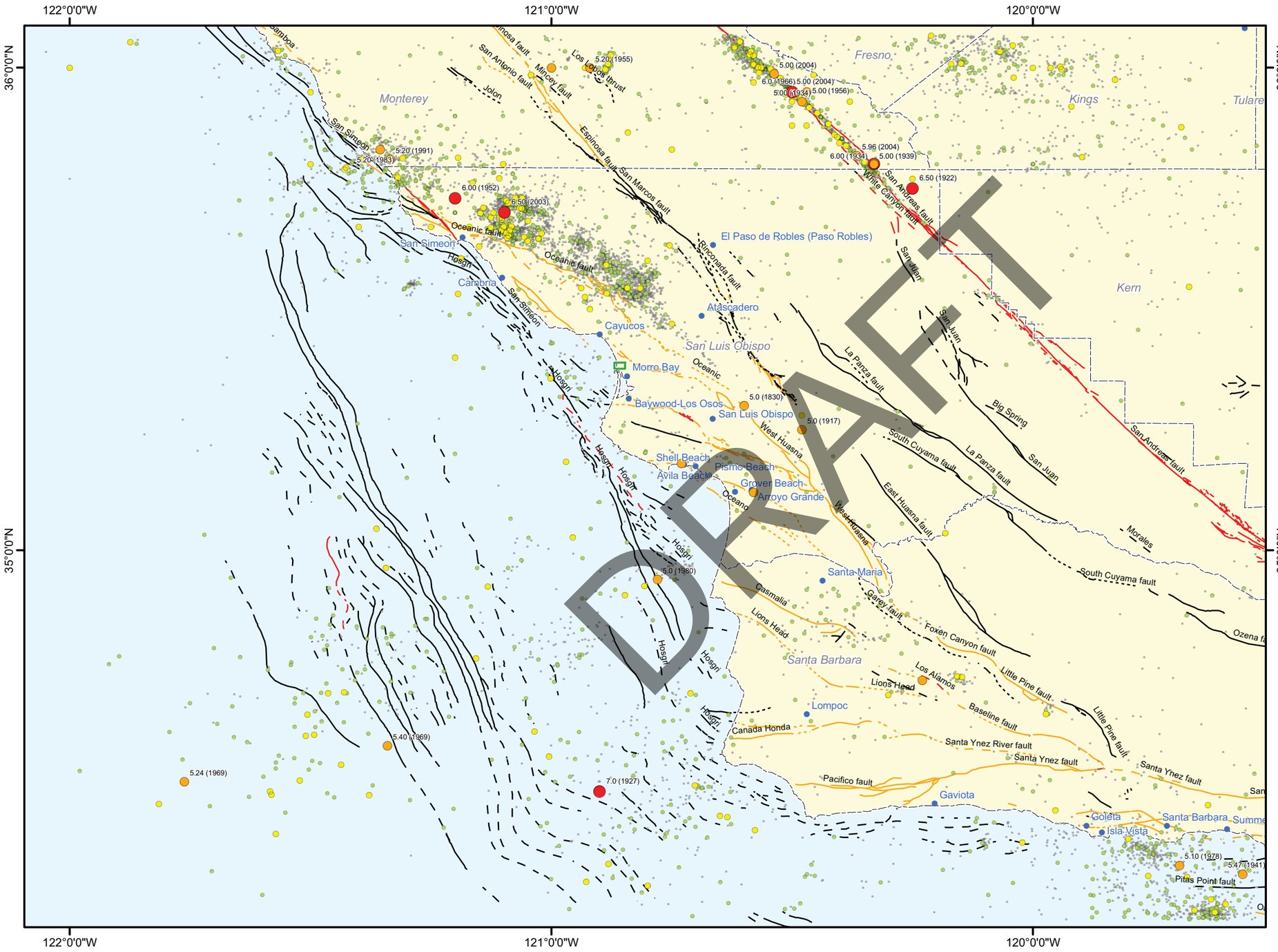
x 6193 Megafossil locality - U.C.L.A. locality number

Vollmer Ranch name/property owner

REGIONAL GEOLOGIC MAP

Morro Bay - Cayucos WWTP Upgrade Project
160 Atascadero Road
Morro Bay, California

PLATE 3



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Legend

Project Vicinity

Earthquake Magnitude
Magnitudes equal to and greater than 5 are labeled.

- 2.0 - 2.9
- 3.0 - 3.9
- 4.0 - 4.9
- 5.0 - 5.9
- >6.0

Faults (dashed where inferred, dotted where concealed)

- Active Fault
- Potentially Active Fault
- Inactive Fault

Source:

1) Earthquake Data:
   Earthquake epicenters from:
   a) ANSS Composite
      Catalog Search, 1933 to 2008,
      <www.ncedc.org/anss/> (downloaded March 2008)
   b) "Seismotectonic framework, coastal central California", Seismotectonics of the Central California Coast Range, Special Paper 292, Geological Society of America, 1994.

2) Faults:
   a) Bryant, 2005
   b) Jennings, 1994

N

0 7.5 15 30 Miles

HISTORICAL SEISMICITY MAP
Morro Bay – Cayucos WWTP Upgrade Project
Morro Bay, California

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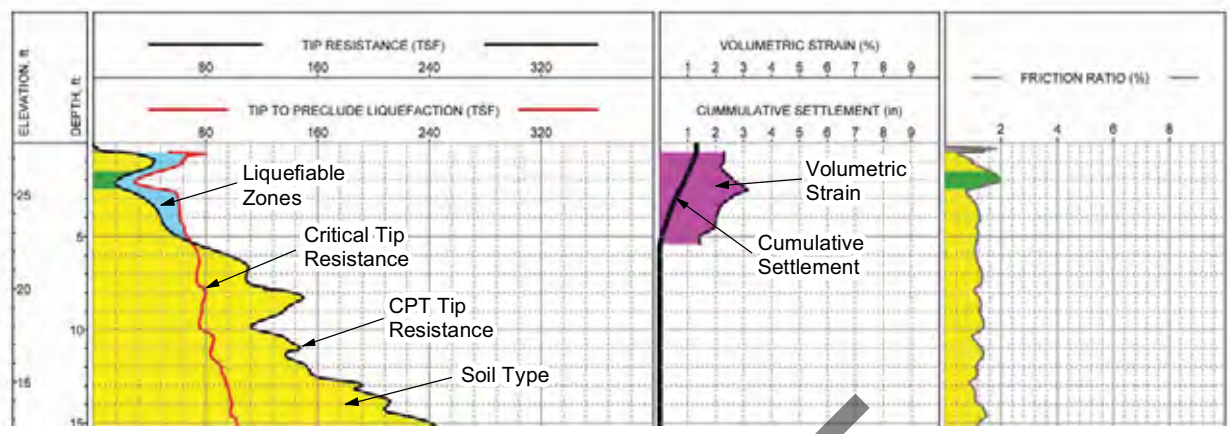
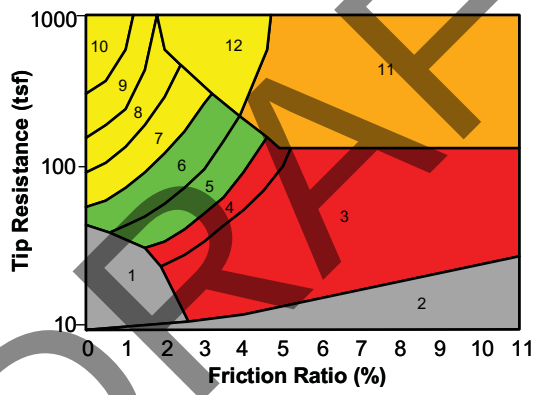


COLOR LEGEND FOR FRICTION RATIO TRACES



Zone	Soil Behavior Type	U.S.C.S.
1	Sensitive Fine-grained	OL-CH
2	Organic Material	OL-OH
3	Clay	CH
4	Silty Clay to Clay	CL-CH
5	Clayey Silt to Silty Clay	MH-CL
6	Sandy Silt to Clayey Silt	ML-MH
7	Silty Sand to Sandy Silt	SM-ML
8	Sand to Silty Sand	SM-SP
9	Sand	SW-SP
10	Gravelly Sand to Sand	SW-GW
11	Very Stiff Fine-grained *	CH-CL
12	Sand to Clayey Sand *	SC-SM

*overconsolidated or cemented

CPT CORRELATION CHART

(Robertson and Campanella, 1988)

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KEY TO LIQUEFACTION LOGS

Morro Bay – Cayucos WWTP Upgrade Project
Morro Bay, California

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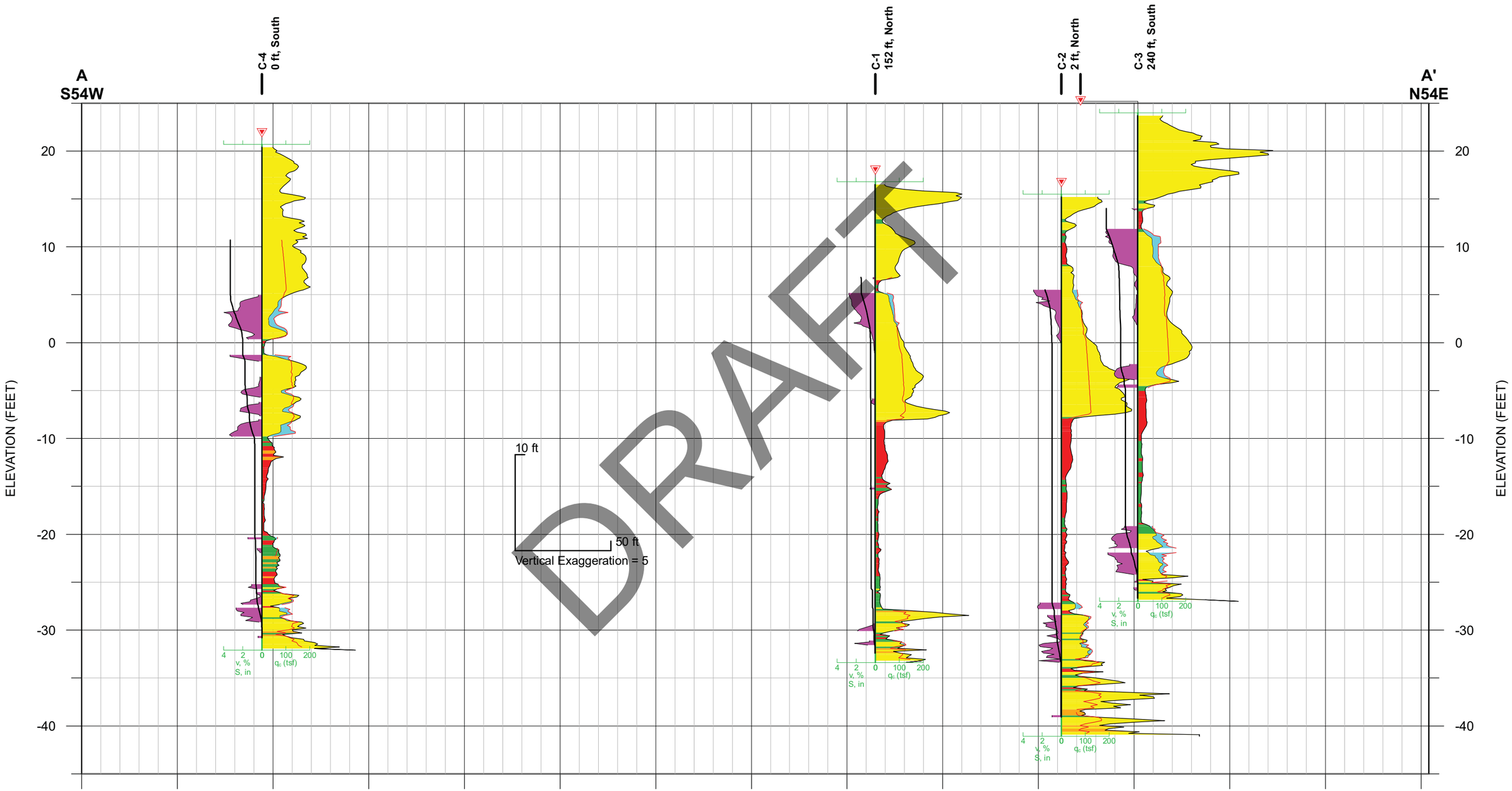

LIQUEFACTION CROSS SECTION A-A'
M = 7.34, P.G.A. = 0.34 g
Morro Bay – Cayucos WWTP Upgrade Project
Morro Bay, California

PLATE 6

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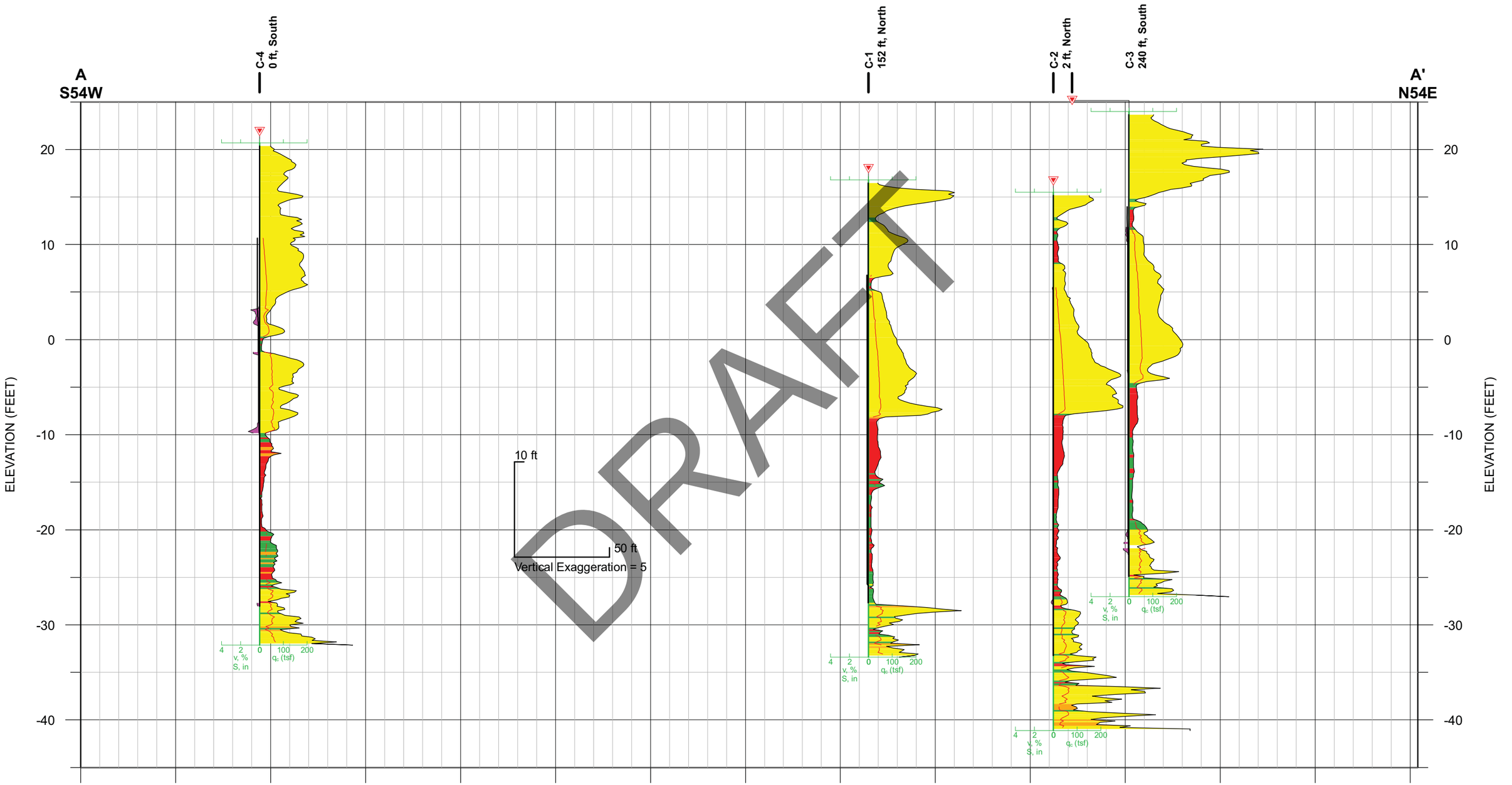

LIQUEFACTION CROSS SECTION A-A'
M = 6.5, P.G.A. = 0.15 g
Morro Bay – Cayucos WWTP Upgrade Project
Morro Bay, California

PLATE 7

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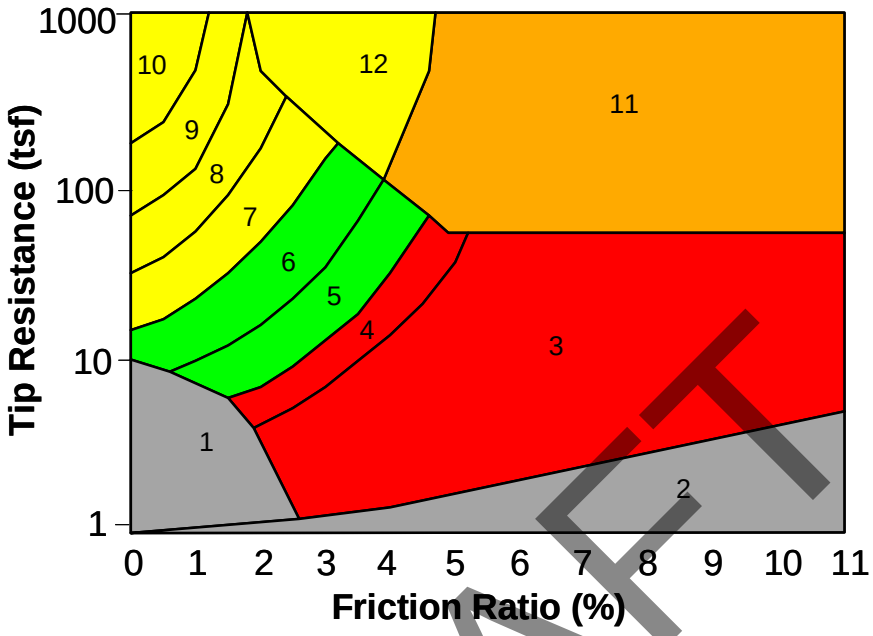
APPENDIX A

COLOR LEGEND FOR FRICTION RATIO TRACES



Zone	Soil Behavior Type	U.S.C.S.
1	Sensitive Fine-grained	OL-CH
2	Organic Material	OL-OH
3	Clay	CH
4	Silty Clay to Clay	CL-CH
5	Clayey Silt to Silty Clay	MH-CL
6	Sandy Silt to Clayey Silt	ML-MH
7	Silty Sand to Sandy Silt	SM-ML
8	Sand to Silty Sand	SM-SP
9	Sand	SW-SP
10	Gravelly Sand to Sand	SW-GW
11	Very Stiff Fine-grained *	CH-CL
12	Sand to Clayey Sand *	SC-SM

*overconsolidated or cemented

CPT CORRELATION CHART
(Robertson and Campanella, 1988)

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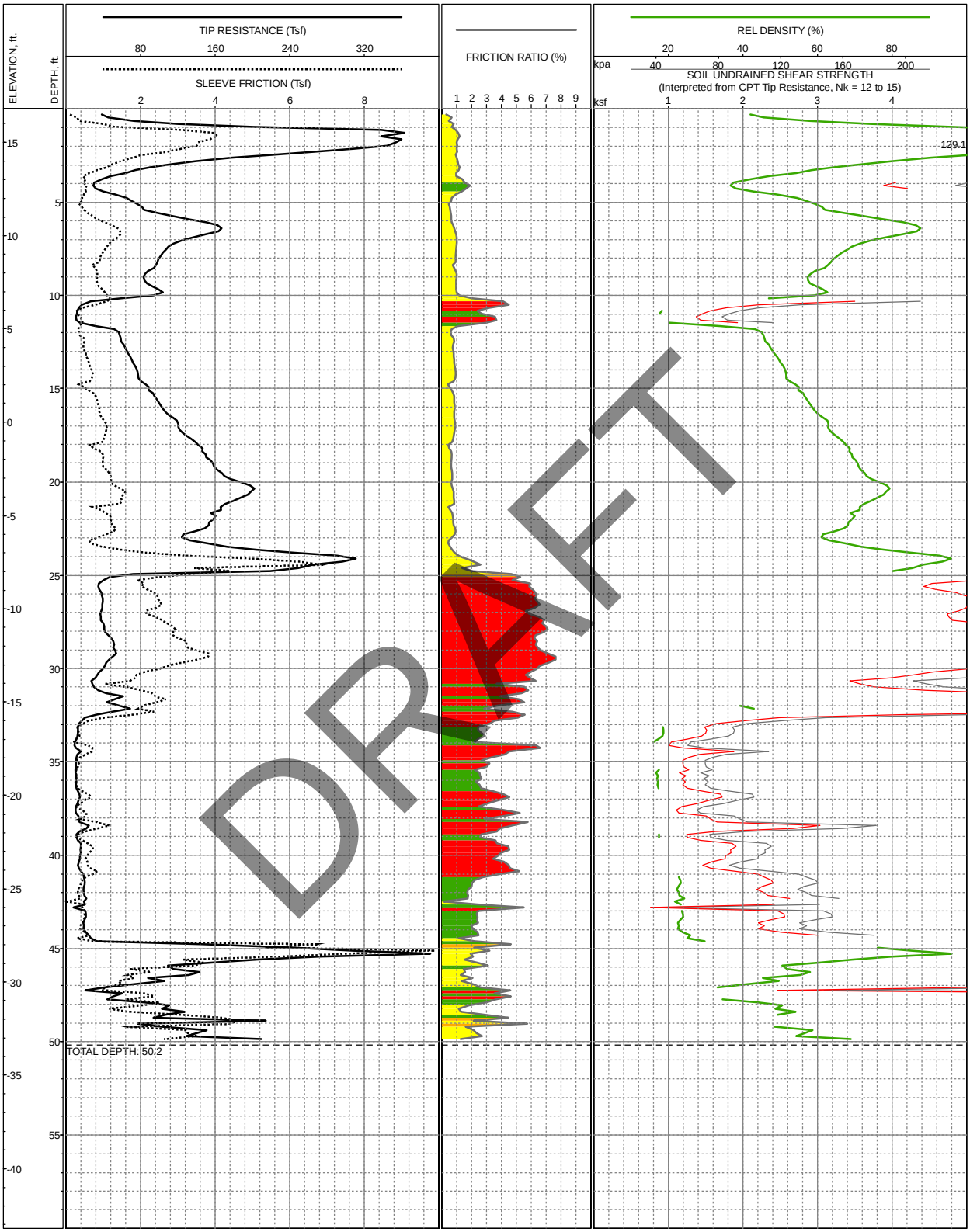
KEY TO CPT LOGS

Morro Bay – Cayucos WWTP Upgrade Project
Morro Bay, California



LOCATION: Approx. 25' East of Existing Storage Shed
SURFACE EL: 16.8ft +/- (NAVD88)
COMPLETION DEPTH: 50.2ft
TESTDATE: 10/16/2009

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Earth Systems Pacific
REVIEWED BY: J Blanchard

LOG OF CPT NO: C-1

Morro Bay – Cayucos WWTP Upgrade Project
Morro Bay, California

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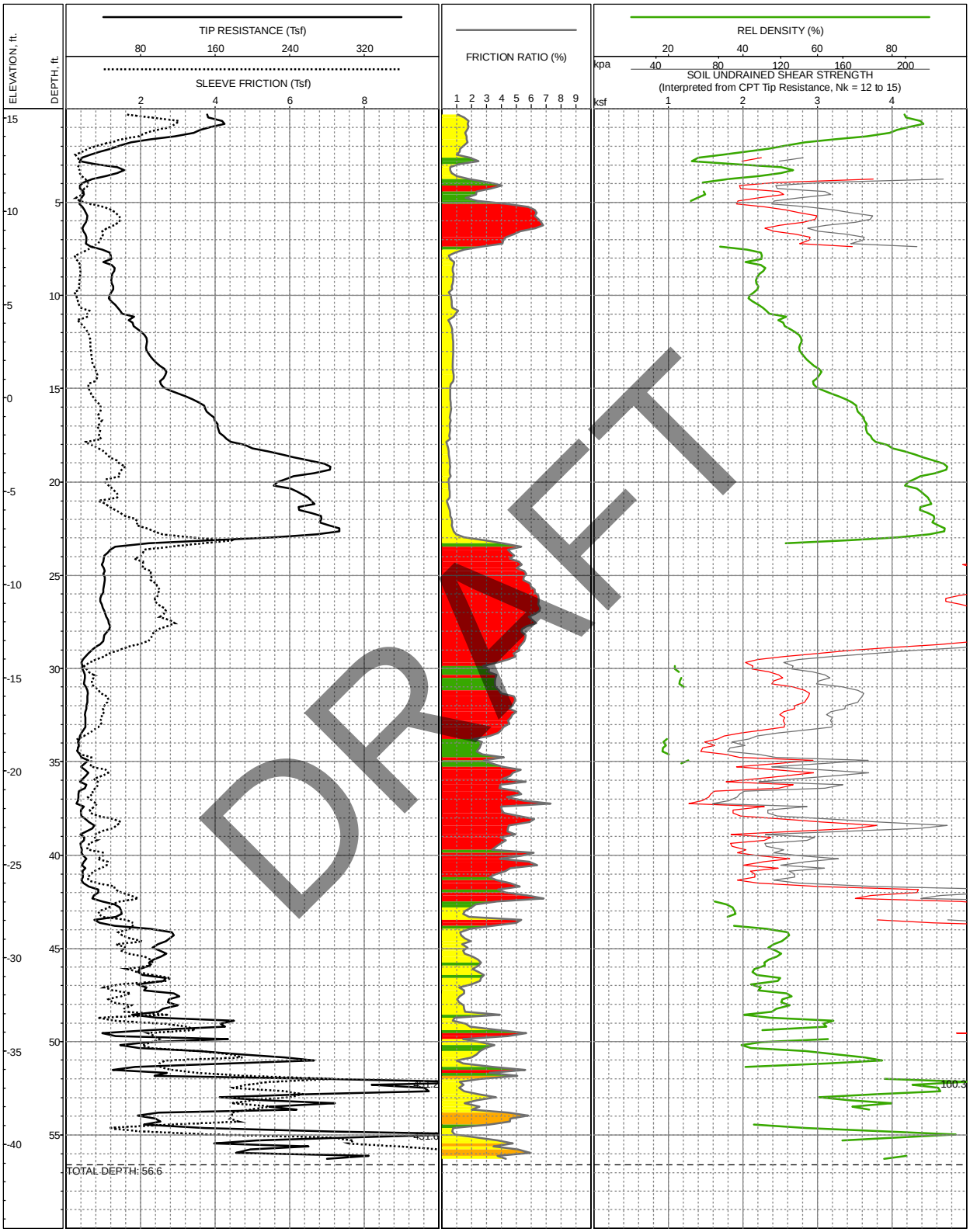

LOCATION: Approx. 25' North of Preliminary IPS Location
SURFACE EL: 15.5ft +/- (NAVD88)
COMPLETION DEPTH: 56.6ft
TESTDATE: 10/16/2009

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Earth Systems Pacific
REVIEWED BY: J Blanchard

LOG OF CPT NO: C-2

Morro Bay – Cayucos WWTP Upgrade Project
Morro Bay, California

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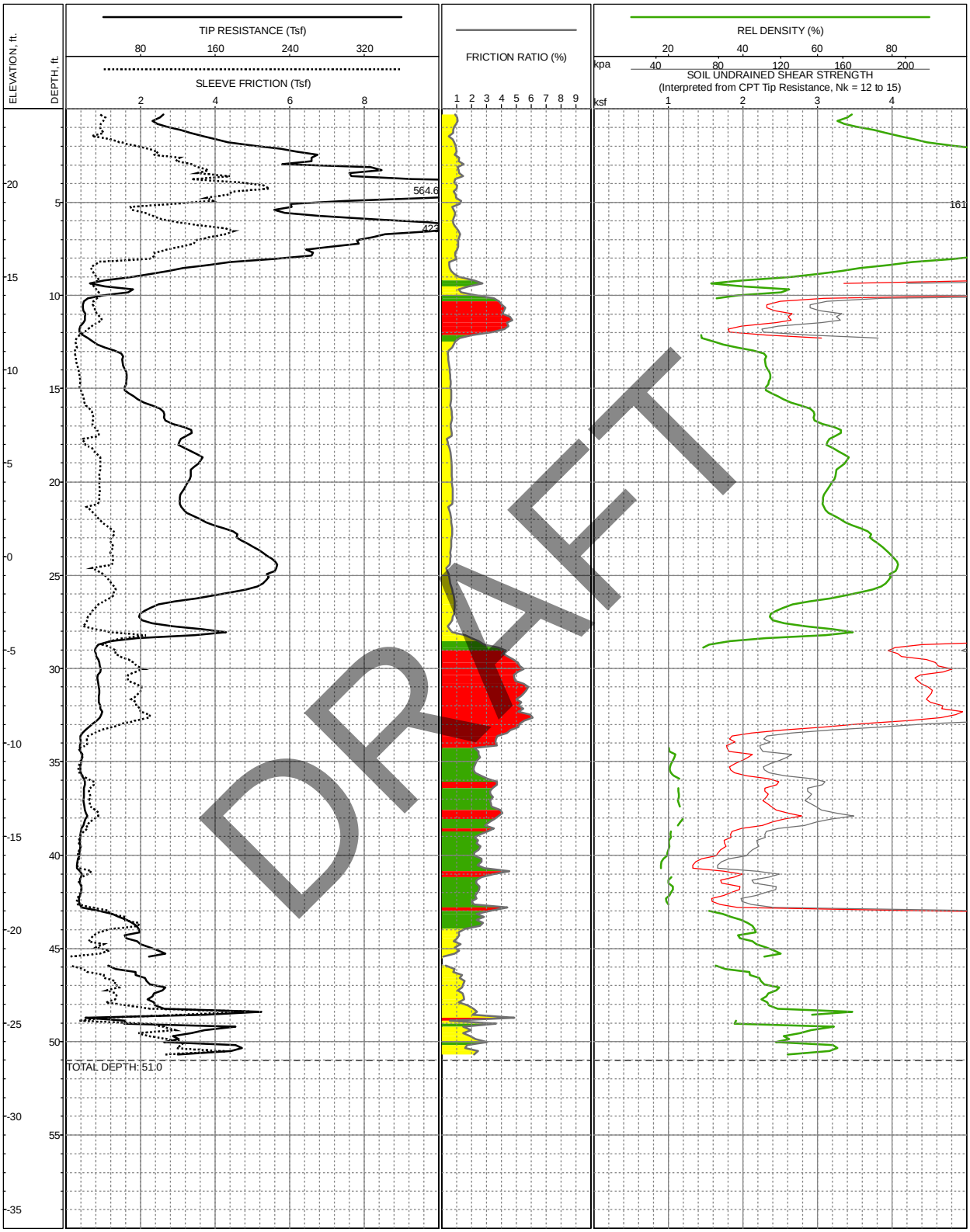

LOCATION: approx. 15' East of Preliminary Oxidation ditches Location
SURFACE EL: 24.0ft +/- (NAVD88)
COMPLETION DEPTH: 51.0ft
TESTDATE: 10/16/2009

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Earth Systems Pacific
REVIEWED BY: J Blanchard

LOG OF CPT NO: C-3

Morro Bay – Cayucos WWTP Upgrade Project
Morro Bay, California

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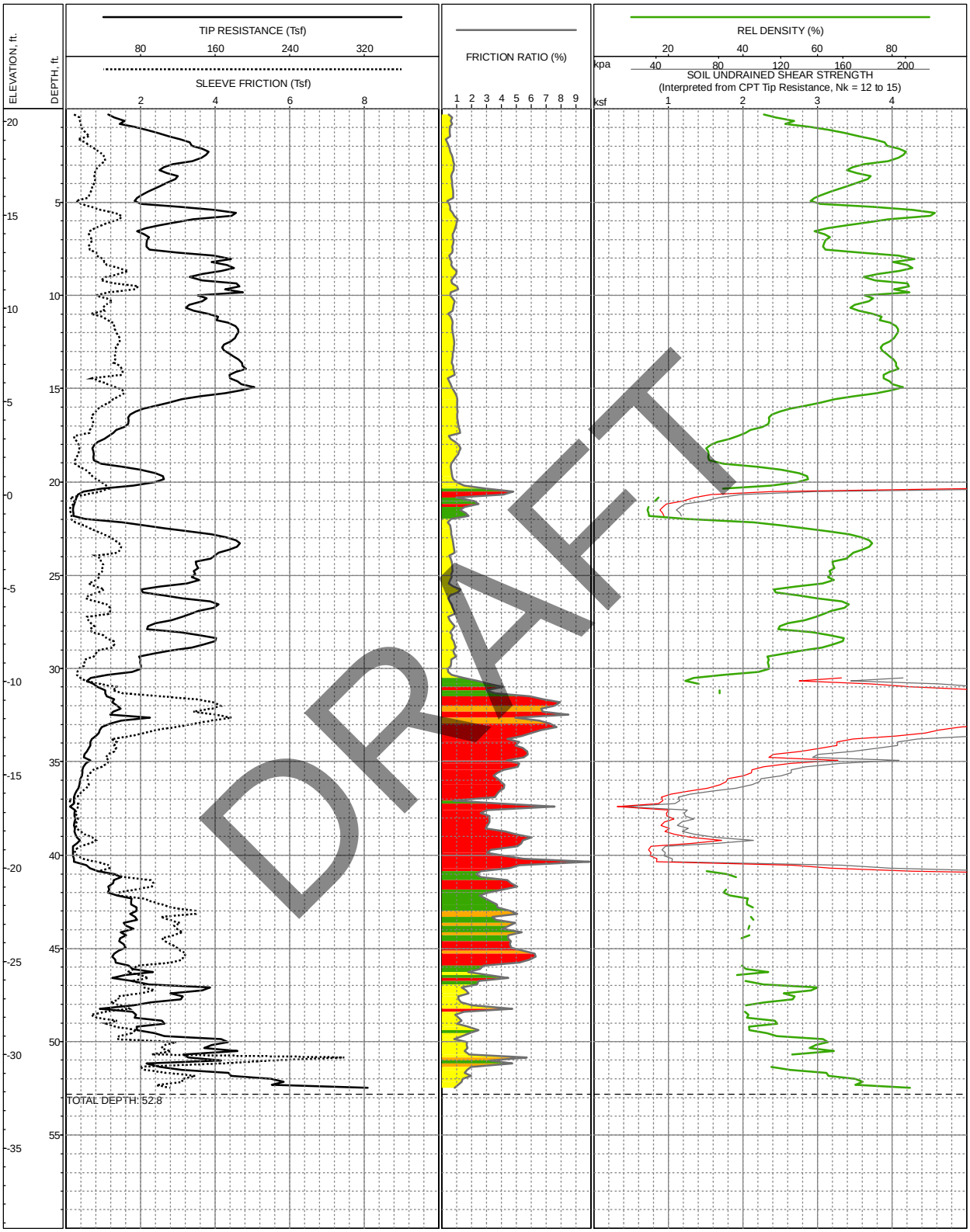

LOCATION: Area of Preliminary Secondary Clarifier Location
SURFACE EL: 20.7ft +/- (NAVD88)
COMPLETION DEPTH: 52.8ft
TESTDATE: 10/16/2009

EXPLORATION METHOD: Cone Penetrometer
PERFORMED BY: Earth Systems Pacific
REVIEWED BY: J Blanchard

LOG OF CPT NO: C-4

Morro Bay – Cayucos WWTP Upgrade Project
Morro Bay, California

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APPENDIX B

Morro Bay WWTP
160 Atascadero Road
Morro Bay, CA 93442

Inquiry Number: 2728018.2
March 29, 2010

The EDR Aerial Photo Decade Package

DRAFT

EDR® Environmental Data Resources Inc

440 Wheelers Farms Road
Milford, CT 06461
800.352.0050
www.edrnet.com

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Date EDR Searched Historical Sources:

Aerial Photography March 29, 2010

Target Property:

160 Atascadero Road

Morro Bay, CA 93442

Year	***Scale***	***Details***	***Source***
1937	Aerial Photograph. Scale: 1"=555'	Flight Year: 1937	Army
1949	Aerial Photograph. Scale: 1"=555'	Flight Year: 1949	Aero
1956	Aerial Photograph. Scale: 1"=555'	Flight Year: 1956	Hycon
1963	Aerial Photograph. Scale: 1"=333'	Flight Year: 1963	Mark Hurd
1972	Aerial Photograph. Scale: 1"=341'	Flight Year: 1972	Mark Hurd
1989	Aerial Photograph. Scale: 1"=666'	Flight Year: 1989	USGS
1994	Aerial Photograph. Scale: 1"=666'	Flight Year: 1994	USGS
2002	Aerial Photograph. Scale: 1"=666'	Flight Year: 2002	USGS
2005	Aerial Photograph. 1" = 604'	Flight Year: 2005	EDR

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YEAR: 1937
= 555'
N

DRAFT
INQUIRY #: 2728018.2
YEAR: 1949
= 555'
N

DRAFT
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YEAR: 1956
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DRAFT
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YEAR: 1972
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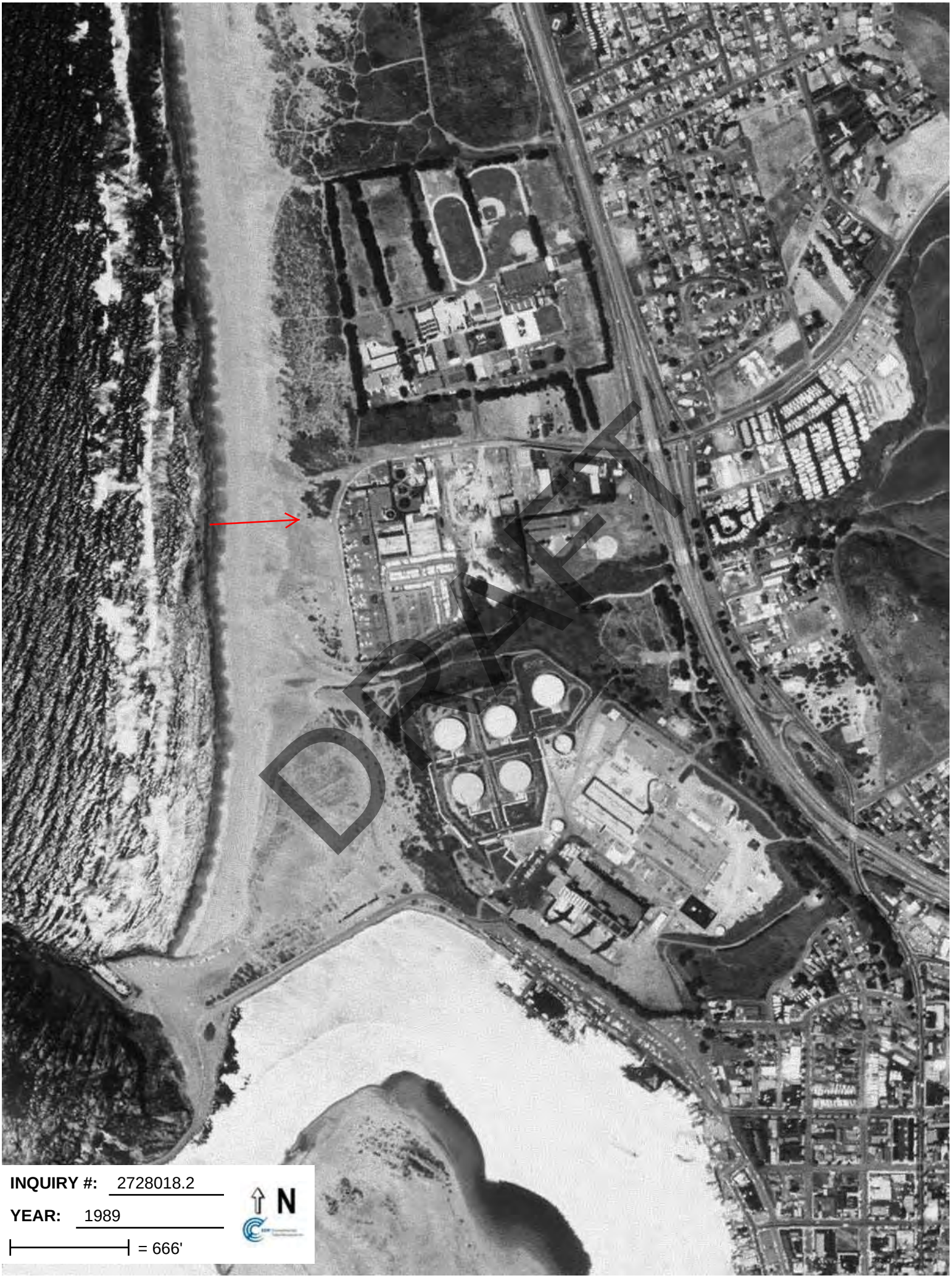
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= 666'
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YEAR: 2005
= 604'
N

Morro Bay WWTP
160 Atascadero Road
Morro Bay, CA 93442

Inquiry Number: 2728018.1
March 25, 2010

The EDR Historical Topographic Map Report

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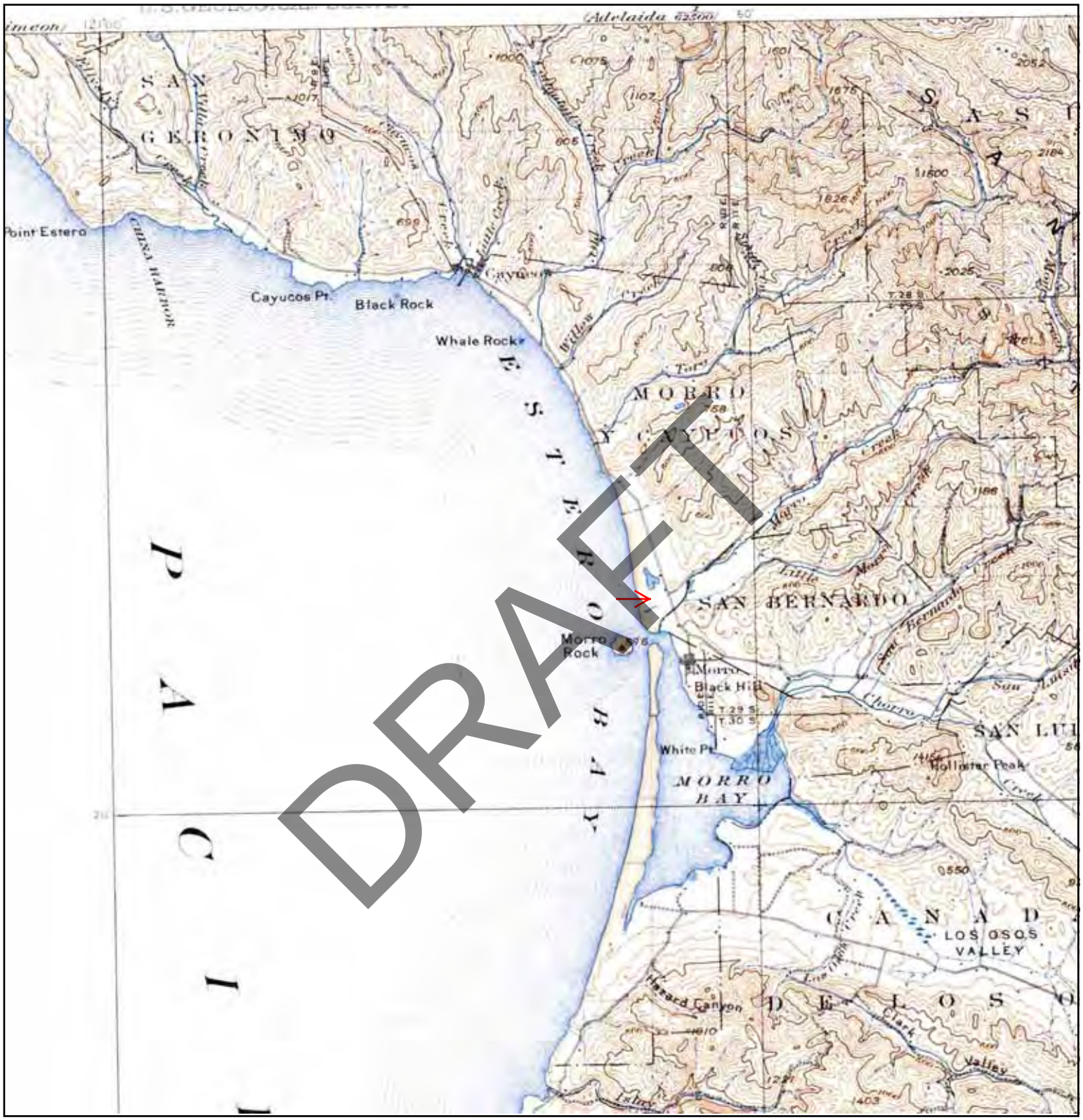
Historical Topographic Map



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Historical Topographic Map

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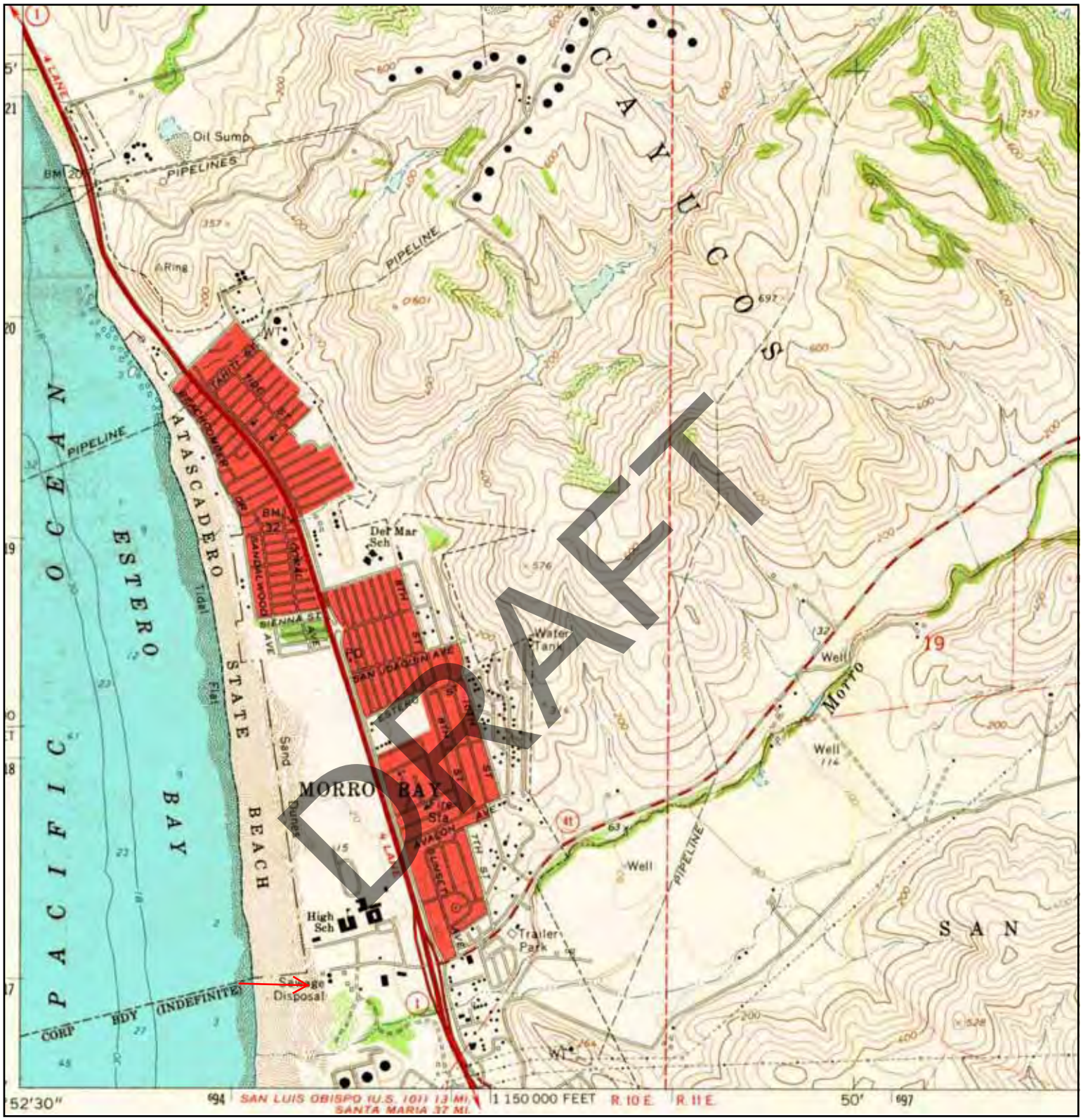
Historical Topographic Map



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Historical Topographic Map

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Historical Topographic Map

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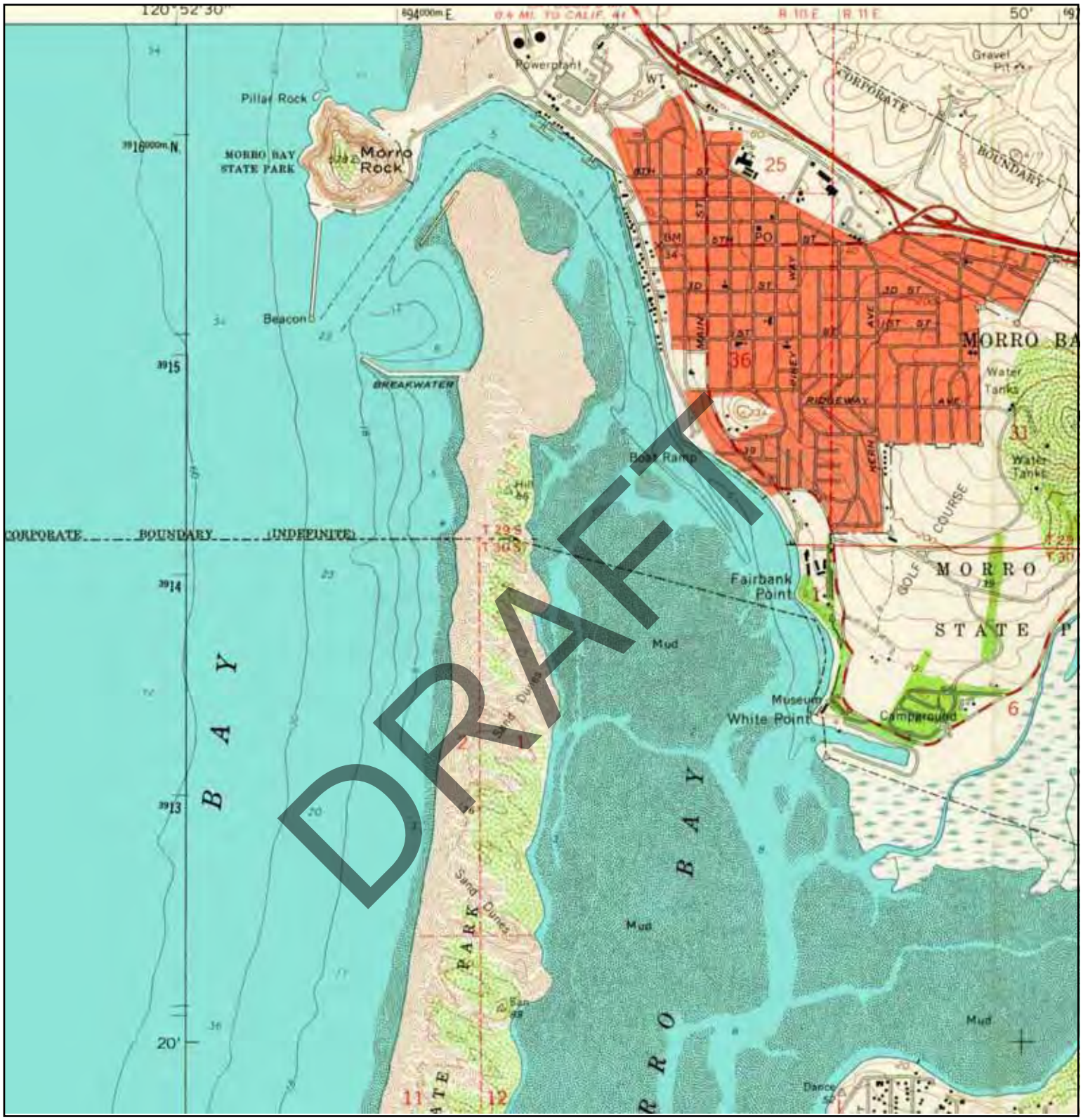
Historical Topographic Map



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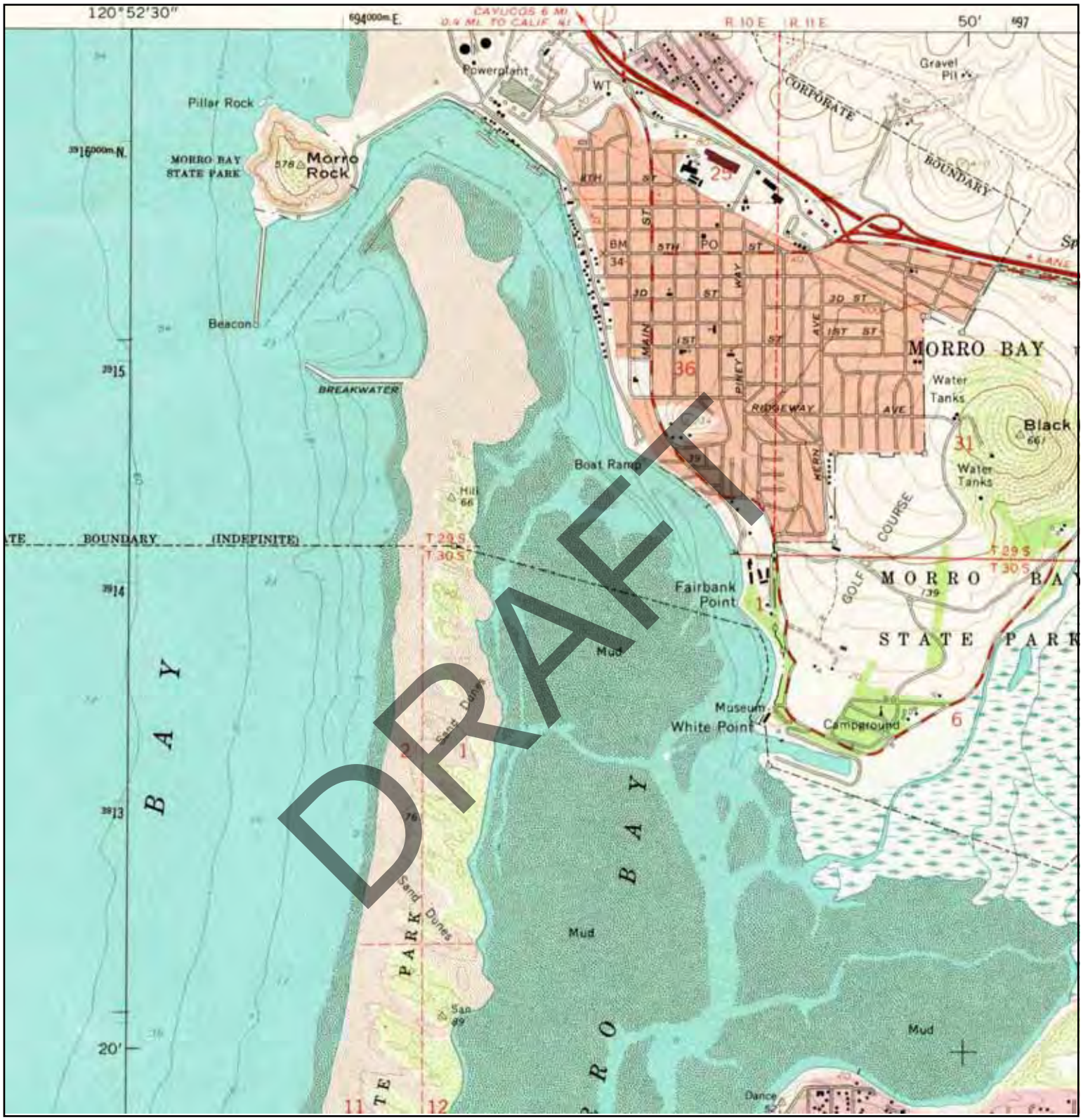
Historical Topographic Map



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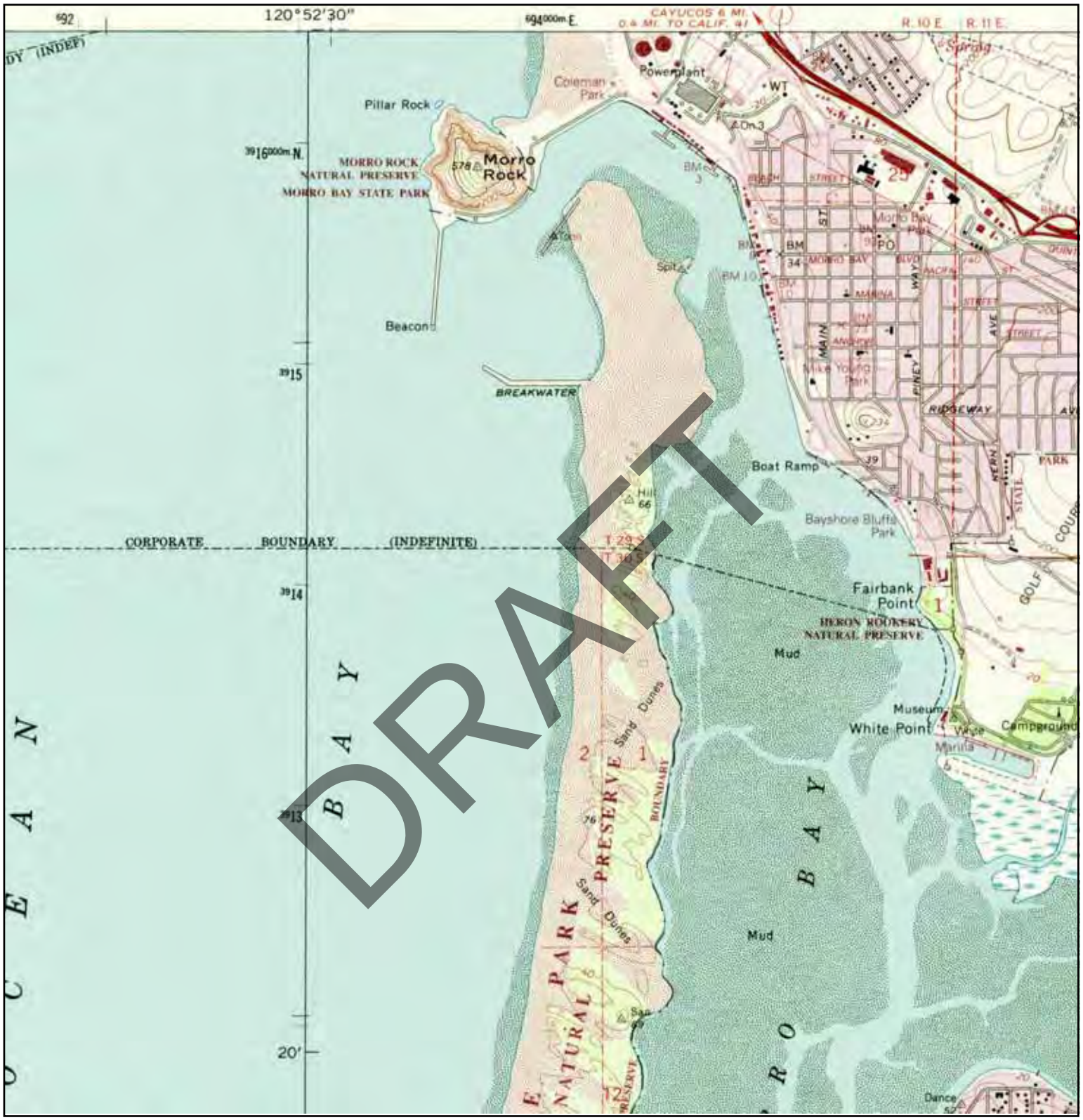
Historical Topographic Map



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APPENDIX C

TF/CCB/Title 22 Considerations TM

M E M O R A N D U M

MWH

To:	Bruce Keogh Bill Callahan Dylan Wade	**Date:**	June 16, 2010
From:	Steve Hyland	**Client:**	MBCSD
Subject:	MBCSD WWTP Upgrade TF/CCB/Title 22 Considerations	**Reference:**	1008613.020101

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Introduction

The current scope of work for the City of Morro Bay and Cayucos Sanitary District (MBCSD) Wastewater Treatment Plant (WWTP) Upgrade Project includes one Tertiary Filter (TF), one Chlorine Contact Basin (CCB) for effluent disinfection prior to discharge to the existing ocean outfall, one Chemical Station for sodium hypochlorite and sodium bisulfite, and associated gates, piping, and valves.

The design of these facilities is also intended to include provisions to accommodate the future installation of reclamation capability to meet California Title 22 requirements for "disinfected tertiary recycled water" for unrestricted use. Disinfected tertiary recycled water for unrestricted use is suitable for irrigation of food crops, parks and playgrounds, school yards, residential landscaping, and golf courses, i.e., "purple pipe" applications. Although the current scope of work includes provisions for the design of tertiary filtration, this scope does not include provisions for the design of Title 22 compliant facilities.

The objective of this Technical Memorandum (TM) is to present a conceptual level evaluation of the components and incremental construction costs to implement potential improvements to the proposed TF/CCB facilities. This evaluation examines different combinations of the number and sizes of TFs and CCBs that can be implemented to minimize operation and maintenance activities, improve disinfection efficiency, and position MBCSD for installation of Title 22 reclaimed water capability. Discussion and evaluation of these options are presented below for review and consideration by MBCSD.

Background Information

Initial Reclaimed Water Demand. The initial phase of future reclaimed water facilities would be anticipated to be designed as a minimum for the following reclaimed water demands:

- Utility water for plant needs
- Irrigation of landscaping at the WWTP perimeter (fence line)
- Possible irrigation of the Morro Bay High School grounds
- Truck-fill station for off-site uses.

A preliminary estimate of the reclaimed water demand for the above uses would range from 100 to 500 gpm depending upon time of day.

Note that to achieve "disinfected tertiary recycled water" under Title 22 requirements, the plant effluent must be filtered with technology that has been pilot tested for approval by the California Department of Public Health. The tertiary filter technology discussed below has been approved for Title 22 applications. In addition, the filtered effluent must be disinfected to meet strict bacteriological standards. Where chlorine disinfection is used as proposed for this WWTP Upgrade Project, a modal contact time of at least 90 minutes in the CCB must be provided at peak flow to meet the Title 22 requirements for "disinfected tertiary recycled water". This design peak flow is assumed to be two times the design average flow for Title 22 disinfection.

Tertiary Filters. The tertiary filter technology assumed for this evaluation is cloth media filtration as presented in the draft FMP and draft FMP - Amendment No. 1. Cloth filters consist of rotating disks covered with a pile fabric that filters suspended solids as the flow passes from the exterior face to the interior. The filtrate is carried away by the hollow center shaft that supports the disks. At periodic intervals the disks are rotated and backwashed to clean the cloth media.

The cloth filter units can be provided in 2 disk increments ranging from 2 disks to 12 disks per unit. The average flow capacity rating for cloth filters is based on a nominal hydraulic loading rate of 3 gpm/sf of disk surface or approximately 0.5 mgd per pair of disks. The peak flow capacity of the cloth filters is 6.0 gpm/sf or 1.0 mgd per pair of disks for a 3-hour duration as recommended by the manufacturer. For Title 22 applications, the cloth filters have been approved at the same peak hydraulic capacity of 6.0 gpm/sf. The 3-hour duration is established to preserve the expected cloth media life of 6 to 7 years before replacement. Note the current cost of cloth media replacement is approximately $3000 per pair of disks (not including installation costs).

The cloth filters can be operated at higher flow rates based on a solids loading limitation of 3.25 lb total suspended solids/sf/day. For example, for an effluent quality of 30 mg/L TSS, the corresponding flow rate would be 1.4 mgd per pair of disks. At this loading the filter would be in continuous backwash.

The frequency and duration of backwash is the major factor that determines the cloth media life expectancy. So, extended operation of the cloth filters at the peak hydraulic and solids loading

rates is not desirable. For the purposes of this evaluation, the tertiary filters are assumed to operate at their design average flow capacity and their peaking capability is assumed to be adequate to handle the diurnal variations associated with daily flow that does not exceed the design average flow capacity.

For the purposes of this evaluation, costs for cloth filter units were developed based on pricing provided by the manufacturer (Aqua-Aerobics email dated June 1, 2010). A summary of the estimated construction costs for 2-Disk, 4-Disk, and 6-Disk units are presented in **Table A**.

Table A – Cloth Filter Costs

Description	**2-Disk**	**4-Disk**	**6-Disk**
Capacity (Average Flow)	0.5 mgd	1.0 mgd	1.5 mgd
Cloth Filter Cost	$275,000	$323,000	$371,000
Cloth Filter Cost Installed	$358,000	$420,000	$482,000
Associated Piping & Valves	$54,000	$63,000	$72,000
Mechanical Subtotal	$412,000	$483,000	$554,000
Structural	$5,000	$8,000	$10,000
Architectural	$8,000	$13,000	$16,000
Mechanical	$412,000	$483,000	$554,000
HVAC	$0	$0	$0
Electrical	$40,000	$40,000	$40,000
Instrumentation	$35,000	$35,000	$35,000
Subtotal	$500,000	$579,000	$655,000
Multiplier (a)	1.393	1.393	1.393
Estimated Construction Cost	**$700,000**	**$810,000**	**$910,000**

(a) Multiplier includes construction allowances for general conditions, bonds and insurance, mobilization and demobilization, overhead and profit, and contingency.

Note that the difference in estimated construction costs for the 2-disk, 4-disk, and 6-disk units is relatively small. Where multiple cloth filter units are under consideration, provision for the same size units is desirable for interchangeability of operation and maintenance. Consequently, for the purposes of this evaluation all cloth filter units were assumed to be 6-disk units.

Current Concept - Option A1 and Option A2

Option A1 represents the current TF/CCB concept based on the draft Facility Master Plan (FMP) entitled "Wastewater Treatment Plant Facility Master Plan Report" dated September 2007 prepared by Carollo Engineers and the draft FMP – Amendment No. 1 entitled "Facility Master Plan Amendment No. 1" dated August 2009 prepared by Carollo Engineers. Refer to the flow schematic for Option A1 in Figure 1. Under the current scope of work the following facilities would be provided:

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- Tertiary filtration rated for the average peak season dry weather flow (PSDWF) of 1.5 mgd. A cloth filter or equivalent is the current candidate for the tertiary filtration technology as previously discussed. One 6-disk cloth filter rated for 1.5 mgd average flow would be provided. Note that if a TF had been operated at a minimum of 1.5 mgd over the period of record from Y1995 to Y2009, over 96 percent of the annual effluent produced by the WWTP would have received tertiary filtration under this hypothetical operation.

- One CCB sized to provide a hydraulic retention time (HRT) of 60 minutes at the average PSDWF of 1.5 mgd and 16 minutes at the annual peak day flow (PDF) of 5.6 mgd. This would correspond to a minimum hydraulic basin volume of 63,000 gal. The preliminary basin geometry to provide a length to width ratio of 40:1 and depth to width ratio of 1:1 would be approximately 83 feet long by 23 feet wide. This basin geometry includes allowances for wall thicknesses and assumes a 3-pass configuration with channels that have a 6-feet width and 6-feet side water depth.

- Utility Water Pump Station would be provided to deliver treated effluent for ancillary plant uses such as spray water and seal water for mechanical equipment and pumps. The provision for utility water offsets the alternate use of potable water to provide these functions.

- Chemical Station would be provided with storage tanks, feed pumps, piping, valves, controls, and appurtenances to deliver sodium hypochlorite (bleach) and sodium bisulfite for disinfection and dechlorination, respectively. Note that the Chemical Station is assumed to be the same for all options.

Option A2 represents the second phase of the current concept of Option A1 with the addition of future Title 22 facilities. Under this scenario the default approach to provide future Title 22 facilities is the addition of a second TF and a second CCB that match the size of the TF and CCB provided in Option A1. Refer to the flow schematic of Option A2 in Figure 2. In addition, a Reclaimed Water Pump Station would be added in parallel to the Utility Water Pump Station for delivery to meet the initial reclaimed water demands previously described.

With the Option A2 facilities, the plant would operate two TF/CCB trains in parallel. One TF/CCB train would operate as under Option A1 for ocean discharge with capacity for an average flow of 1.5 mgd and a peak day flow of 5.6 mgd. The other TF/CCB train would operate to meet Title 22 requirements. The reclaimed water capacity of one TF/CCB train would be based on the 90-minute HRT in the CCB required for Title 22 disinfection. This corresponds to a peak flow rate of 1.0 mgd and an average flow rate of 0.5 mgd.

Note that the TF capacity needed for the Title 22 capacity of 0.5 mgd could be achieved with a 2-disk filter; but as previously discussed, the estimated construction cost difference of approximately $200,000 is relatively small and is outweighed by the benefits of having equal size units for interchangeability.

The estimated construction costs for Option A1 and Option A2 are presented in **Table B**.

Table B – Option A1 and Option A2 Estimated Construction Costs

Description	Amount
Option A1	
Tertiary Filter	$910,000
One-chamber CCB	$428,000
UW Pump Station	$399,000
Chemical Station	$506,000
Subtotal	$2,243,000
Option A2	
Tertiary Filter	$910,000
One-chamber CCB	$428,000
UW Pump Station	$0
RW Pump Station	$399,000
Chemical Station	$50,000
Subtotal	$1,737,000
Option A Total	**$4,030,000**

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Alternative Concept - Option B1 and B2

Option B1 is a refinement of Option A1 that provides the same TF unit (6-disk cloth filter), Utility Water Pump Station, and Chemical Station, but provides a CCB with two chambers in lieu of one chamber to improve operation and maintenance at a relatively small increase in estimated construction cost. MBCSD and MWH staff discussed the difficulty of operating and maintaining a single chamber CCB at a coordination meeting conducted on April 26, 2010. Two CCB chambers are desirable so that one CCB chamber can be taken out of service for inspection and cleaning while the other chamber remains in service to provide disinfection. The use of a single chamber CCB is currently employed at the existing WWTP and this situation has created operation and maintenance difficulties in the past. For example, the existing CCB utilizes a chain and flight mechanism to sweep solids that accumulate on the floor to one end of the basin for disposal. The CCB has to be periodically taken out of service and drained to inspect and service the chain and flight mechanism. Plant effluent coliform violations have occurred in these instances.

A two-chamber CCB with an equivalent volume as the baseline CCB in Option A was initially evaluated. Refer to the flow schematic of Option B1 in Figure 3. The CCB was divided to provide two chambers with half the overall volume in each chamber. In order to provide comparable length to width and depth to width ratios, the preliminary basin geometry would be approximately 69 feet long by 38.5 feet wide. This basin geometry includes allowances for wall

thicknesses and assumes a 3-pass configuration in each chamber with channels that have a 5-feet width and 5-feet side water depth.

The major differences in the structural elements of the CCBs for Option A and Option B are summarized in **Table B**. The additional estimated construction cost to provide a two-chamber CCB with the same overall HRT as Option A is approximately $180,000. A two-chamber CCB is recommended to be included in the final design.

Table C – CCB

Description	Unit	One-Chamber CCB	Two-Chamber CCB
Floor slab	sf	1910	2660
Hydraulic walls	lf	212	275
Divider Walls	lf	148	244

Upon further examination of the two-chamber CCB, the cost to increase the relatively shallow side water depth of 5 feet to 8 feet was evaluated. Although the depth to width ratio would deviate from the desired 1:1, increasing the depth does not alter the more critical length to width ratio. The increase in side water depth from 5.0 feet to 8.0 feet would increase the physical volume of each CCB chamber to 59,000 gal. If the disinfection effectiveness of the physical volume is assumed to be reduced to 90% because the depth to width ratio has changed from 1:1 to 1.6:1, then the resulting effective volume would be 53,000 gal in each CCB chamber for the desired modal contact time. The estimated construction cost to increase the side water depth from 5.0 feet to 8.0 feet is approximately $60,000.

The net increase in estimated construction cost to provide a two-chamber CCB with a deeper side water depth is ($180,000 + $60,000 =) $240,000 for Option B1 versus Option A1. The incremental cost to provide the greater flexibility afforded by two chambers and increasing the effective volume to improve disinfection capability (lower chemical cost) is relatively small compared to the overall construction cost of the TF/CCB facilities. Option B1 is recommended as a better value than Option A1.

Option B2 represents the second phase of Option B1 with the addition of Title 22 facilities. Refer to the flow schematic of Option B2 in Figure 4. However, Option B2 does not require the future construction of additional CCB volume. The increase in effective volume obtained by increasing the side water depth under Option B1 provides sufficient volume for both ocean discharge and Title 22 disinfection. Under Option B2 the future facilities would consist of a second TF and a Reclaimed Water Pump Station.

Analogous to the Option A2 scenario, with the Option B2 facilities the plant would operate two TF/CCB trains in parallel. One TF/CCB train would operate similar to Option A1 for ocean discharge but with reduced hydraulic capacity because of the smaller Option B effective volume of 53,000 gal per chamber versus the Option A volume of 63,000 gal. The 53,000 gal CCB chamber volume would provide an effective average flow capacity of 1.3 mgd and peak flow capacity of 4.7 mgd for ocean discharge.

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The other TF/CCB train would operate to meet Title 22 requirements. The reclaimed water capacity of one TF/CCB train would be based on the 90-minute HRT in the CCB required for Title 22 disinfection. The effective volume of 53,000 gal in one chamber can provide 90 minutes of HRT at a peak flow rate of 0.8 mgd or an average flow rate of 0.4 mgd for Title 22 operation.

Under Option B the plant could operate one TF/CCB train for outfall discharge (average flow of 1.3 mgd and peak flow up to 4.7 mgd) and one TF/CCB train for Title 22 (average flow of 0.4 mgd and peak flow up to 0.8 mgd). The combined average flow capacity of the two TF/CCB trains of (1.3 mgd + 0.4 mgd =) 1.7 mgd would exceed the design PSDWF - Average of 1.5 mgd. The combined peak flow capacity of the two TF/CCB trains of (4.7 mgd + 0.8 mgd =) 5.5 mgd would effectively meet the design peak day flow of 5.6 mgd during wet weather events.

The estimated construction costs for Option B1 and Option B2 are presented in **Table D**. The estimated construction costs for Option A1 and Option A2 are included for comparison. Note that the overall cost of Option B is approximately $200,000 lower than Option A.

Table D – Options A1, A2, B1, and B2 Estimated Construction Costs

Description	Amount	Description	Amount
Option A1		Option B1	
Tertiary Filter	$910,000	Tertiary Filter	$910,000
One-chamber CCB	$428,000	Two-chamber CCB	$668,000
UW Pump Station	$399,000	UW Pump Station	$399,000
Chemical Station	$506,000	Chemical Station	$506,000
Subtotal	$2,240,000	Subtotal	$2,480,000
Option A2		Option B2	
Tertiary Filter	$910,000	Tertiary Filter	$910,000
One-chamber CCB	$428,000	CCB	$0
UW Pump Station	$0	UW Pump Station	$0
RW Pump Station	$399,000	RW Pump Station	$399,000
Chemical Station	$50,000	Chemical Station	$50,000
Subtotal	$1,790,000	Subtotal	$1,360,000
Option A Total	**$4,030,000**	**Option B Total**	**$3,840,000**

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Summary

The estimated construction cost and flow capacities of each option evaluated in this TM are summarized in **Table E** for comparison.

Table E – Comparison of Option A and Option B

Description	Unit	Option A			Option B		
		A1	A2	Total	B1	B2	Total
Est. Construction Cost		$2,240k	$1,790k	$4,030k	$2,480k	$1,360k	$3,840k
Plant Capacity							
Average Flow							
Outfall	mgd	1.5	0.0	1.5	1.5 (b)	0.0	1.3
Title 22	mgd	0.0	0.5	0.5	0.0	0.4	0.4
Plant	mgd	1.5	0.5	2.0	1.5 (b)	0.4	1.7
Peak Flow							
Outfall	mgd	5.6	0.0	5.6	5.6 (b)	0.0	4.7
Title 22	mgd	0.0	1.0	1.0	0.0	0.8	0.8
Plant	mgd	5.6	1.0	6.6	5.6 (b)	0.8	5.5

A summary of the basis of design for Option A and Option B is presented in Table F at the end of the TM for reference.

In reviewing the two options presented in this TM, Option B is preferred based on the following considerations:

- Option A1 is the baseline configuration that represents the current scope of design.
- Option A2 represents the default approach to provide Title 22 reclaimed water capability in the future by adding a second TF and second CCB that are the same sized units as provided in Option A1.
- Option B would provide the immediate convenience and flexibility of two CCB chambers in lieu of one chamber and increase the side water depth from 5 feet to 8 feet for a relatively small investment of $240,000.
- The increase the side water depth from 5 feet to 8 feet under Option B would provide an increase in disinfection volume that would increase disinfection efficiency (lower chemical cost) during the first phase of operation (Option B1). The increase in disinfection volume also simplifies and reduces the estimated construction cost to implement Title 22 capability in the future (Option B2).

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- After installation of future Title 22 reclaimed water capability (Option A2 or Option B2), the overall plant capacity provided under Option B more closely matches the design parameters than Option A. Option A capacity would exceed the needed plant design criteria as indicated in Table F.

Table F – Option A and Option B Plant Capacity Comparison

Description	Unit	Option A	Option B	Design Criteria
PSDWF - Average	mgd	2.0	1.7	1.5
Peak Day Flow	mgd	6.6	5.5	5.6

Based on the considerations presented above, Option B is recommended for implementation for the MBCSD WWTP Upgrade Project.

Please contact Steve Hyland if you have any questions.

c. Don Spiegel
Roger Stephenson

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Table F – Basis of Design for Option A and Option B

Description	Unit	Option A			Option B		
		A1	A2	Total	B1	B2	Total
Tertiary Filters							
No.	--	1	1	2	1	1	2
Size	Disks	6-D	6-D	6-D	6-D	6-D	6-D
Installed TF Cap.							
Average Flow	mgd	1.5	1.5	3.0	1.5	1.5	3.0
CCBs							
No. Chambers	--	1	1	2	2		2
Vol. / Chamber (a)	Gal	64,000	64,000	64,000	53,000		53,000
Total Volume (a)	gal	64,000	64,000	128,000	106,000		106,000
Side Water Depth	ft	6	6	6	8		8
Outfall Cap. per Chamber							
Average	mgd	1.5	0.0	1.5	1.3	0.0	1.3
Peak	mgd	5.6	0.0	5.6	4.7	0.0	4.7
Title 22 Cap. per Chamber							
Average	mgd	NA	0.5	0.5	NA	0.4	0.4
Peak	mgd	NA	1.0	1.0	NA	0.8	0.8
UW Pump Station	--	Yes		Yes	Yes		Yes
RW Pump Station	--		Yes	Yes		Yes	Yes
Chemical Station	--	Yes		Yes	Yes		Yes

(a) Volumes somewhat larger than theoretical requirement due to rounding up of structural width and length dimensions. Effective volume for Option B listed.

(b) Actual capacity can be higher with both CCB chambers in service, but values listed are plant design criteria. Disinfection expected to be more efficient (lower chemical costs) in this mode.

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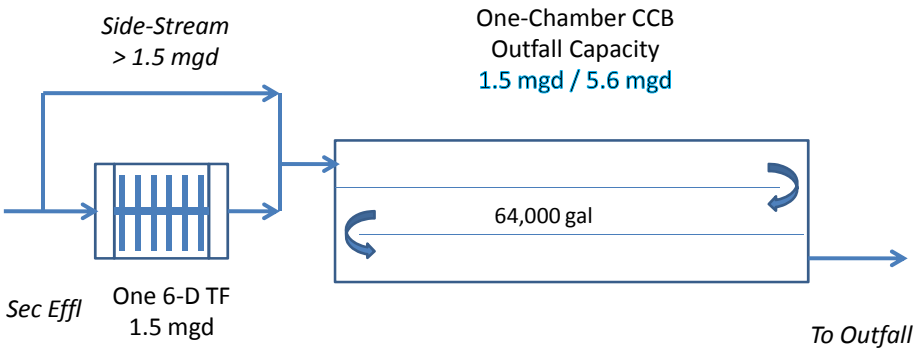
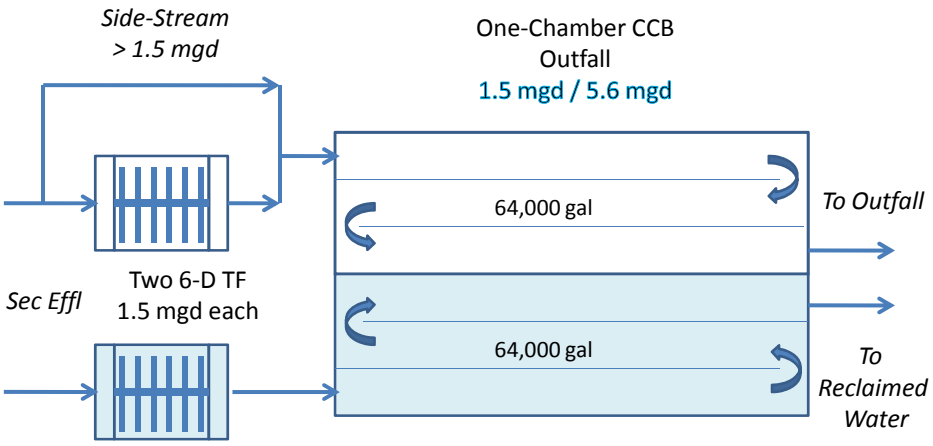
Figure 1 – Option A1 Flow Schematic



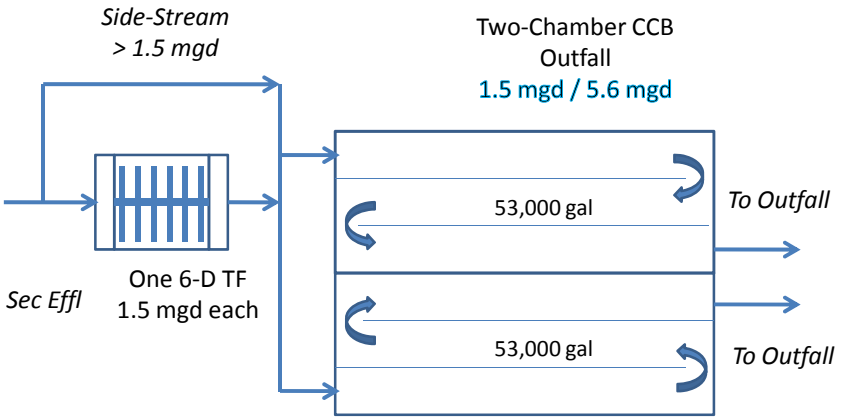
Notes:
1. With a hypothetical 1.5 mgd TF capacity, over 96% of total flow discharged from the WWTP over the period from Y1995 to Y2009 would have received tertiary filtration.
2. Utility Water Pump Station and Chemical Station not shown.

Figure 2 – Option A2 Flow Schematic



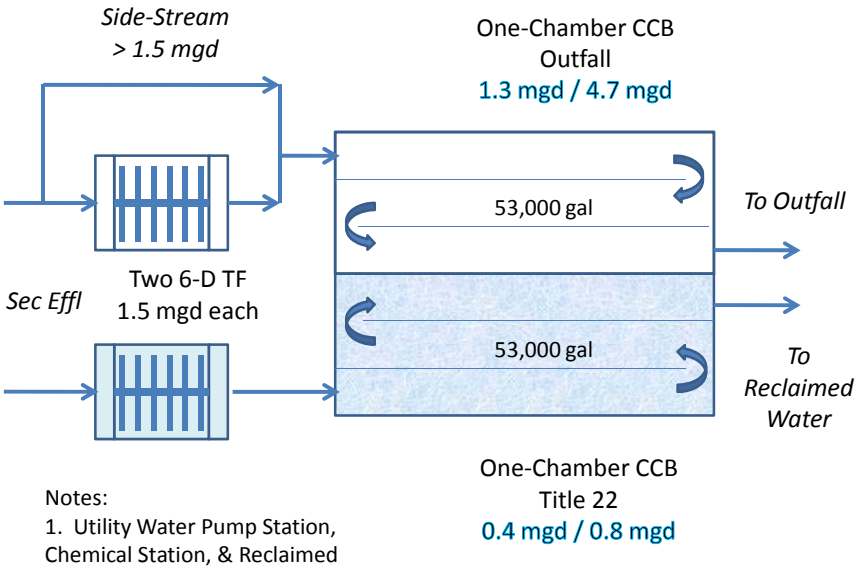
Notes:
1. Utility Water Pump Station, Chemical Station, & Reclaimed Water Pump Station not shown.

Figure 3 – Option B1 Flow Schematic

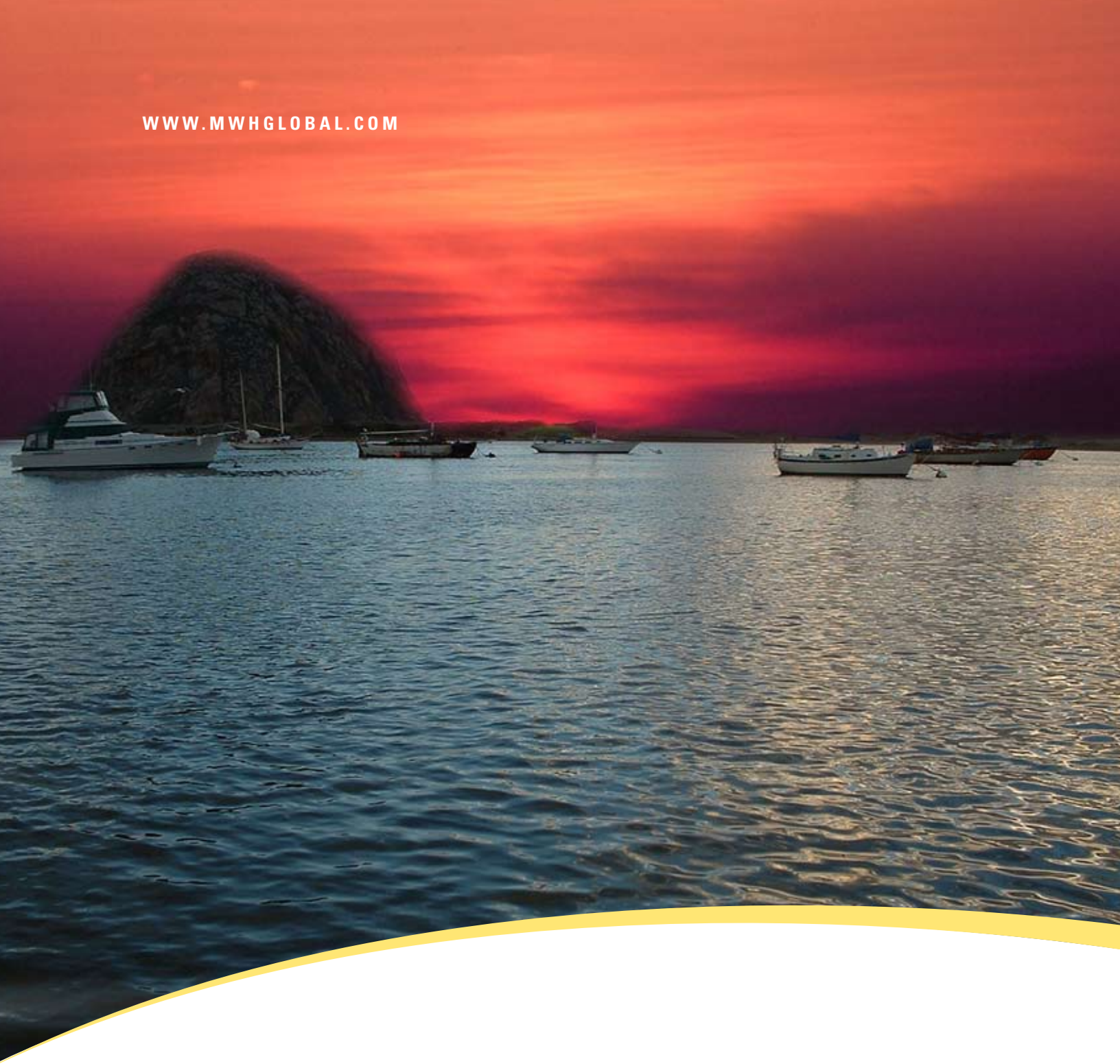


Notes:
1. Utility Water Pump Station & Chemical Station not shown.

Figure 4 – Option B2 Flow Schematic



Notes:
1. Utility Water Pump Station, Chemical Station, & Reclaimed Water Pump Station not shown.



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