APPENDIX 1a

Preliminary Design Report

AECOM

Preliminary Design Report

MSWD
Mission Springs Water District

Mission Springs Water District

West Valley Water Reclamation Facility

December 7, 2018

Table of Contents

List of Figures

List of Tables

Appendices

ACRONYMS

‘	Foot
“	Inch
%	Percent
>	Greater than
ºC	Degrees Celsius
°F	Degrees Fahrenheit
$/mo	Dollars per month
$/kWh	Dollars per kilowatt-hour
AECOM	AECOM Technology Corporation (CONSULTANT)
ACH	Air changes per hour
ACI	American Concrete Institute
ADF	Average daily flow
ADMM	Average Day Maximum Month
ASHRAE	American Society of Heating Refrigeration Air Conditioning Engineers
ASTM	American Society for Testing and Materials
ATAD	Autothermal thermophilic aerobic digestion
AWG	American Wire Gauge
BFP	Belt filter press
bgs	Below ground surface
BOD_5	5-day, biochemical oxygen demand
CAS	Conventional activated sludge
CBC	California Building Code
CEQA	California Environmental Quality Act
cfm	Cubic feet per minute
CFR	Code of Federal Regulations
CHP	Combined heat and power
CIP	Clean in place
CL	Containment liner (epoxy)
CMC	California Mechanical Code
CMDF	Cloth media disc filters
CP	Control panel
CPC	California Plumbing Code
CPE	Comprehensive Plant Evaluation
CRBRWQCB	Colorado River Basin Regional Water Quality Control Board
CS	Concrete hardener/sealer

DBP	Disinfection by-product
DO	Dissolved oxygen
dt	Dry ton
EIR	Environmental Impact Report
ENR	Engineering News Record
F/M	Food to microorganism ratio
FEMA	Federal Emergency Management Agency
fps	Feet per second
ft^2	Square foot
ft/sec	Feet per second
ft/min	Feet per minute
FRP	Fiberglass reinforced plastic
gal	Gallon
GBT	Gravity belt thickener
gpd	Gallons per day
gpd/ft^2	Gallons per day per square foot
gph	Gallons per hour
gpm	Gallons per minute
GWPP	Groundwater Protection Program
H_2S	Hydrogen sulfide
HDPE	High density polyethylene
HP	Horsepower
Hr/wk	Hours per week
HRT	Hydraulic retention time
HVAC	Heating, ventilation and air conditioning
HWWTP	Horton Wastewater Treatment Plant
I&C	Instrumentation and Controls
ICBO	International Conference of Building Officials
IPR	Indirect potable reuse
kWh	Kilowatt-hour
kWh/lb/BOD_5	Kilowatt-hour per pound per biochemical oxygen demand, 5-day
kWh/lb	Kilowatt-hour per pound
kWh/mgal	Kilowatt-hour per million gallons

lb	Pounds
lb/day	Pounds per day
lb/day/sf	Pounds per day per square foot
lb/mo	Pounds per month
lb O_2/day	Pounds of oxygen per day
lb TSS/BOD_5	Pounds of total suspended solids per pounds of biochemical oxygen demand, 5-day
LED	Light-emitting diode
M&E	Metcalf & Eddy
MBR	Membrane bioreactor
MBTU	Millions of British Thermal Units
MCC	Motor control center
MCL	Maximum Contaminant Level
mgal	Millions of gallons
mgal/mo	Millions of gallons per month
mgd	Million gallons per day
mg/L	Milligrams per liter
mg O_2/L/hr	Milligrams of oxygen per liter per hour
mg-min/L	Milligram-minutes per liter
ml/L	Milliliters per liter
ML	Mixed liquor
MLSS	Mixed liquor suspended solids
mm	Millimeters
MOP	Manual of practice
MPN	Most probable number
MSWD	Mission Springs Water District
N/A	Not applicable
NaOH	Sodium hydroxide
NaOCl	Sodium hypochlorite
NEC	National Electric Code
NEMA	National Electrical Manufacturing Association
NFPA	National Fire Protection Association
NH_3-N	Ammonia-nitrogen
NH_4-N	Ammonia-nitrogen
nm	Nanometers
NO_3-N	Nitrate-nitrogen
NPDES	National Pollutant Discharge Elimination System
NPSH	Net positive suction head
NTU	Nephelometric Turbidity Units

O_2	Oxygen
O&M	Operation and maintenance
O/S	Out of service
OSHA	Occupational Safety and Health Administration
OUR	Oxygen uptake rate
ORP	Oxidation reduction potential
pH	Negative log of the hydrogen ion concentration
PE	Polyethylene
PES	Polyethersulfone
PLC	Programmable logic controller
PDR	Preliminary Design Report
PP	Polypropylene
ppm	Parts per million
ppmv	Parts per million by volume
PS	Polysulfone
psf	Per square foot
psi	Pounds per square inch
PTFE	Polytetrafluoroethylene
PVC	Polyvinyl chloride
PVDF	Polyvinylidene fluoride
PWWF	Peak wet weather flow
RAS	Return activated sludge
RDT	Rotary drum thickener
RFP	Request for Proposal
RMA	Running monthly average
rpm	Revolutions per minute
RWC	Recycled water contribution
RWP	Regional wastewater program
SCADA	Supervisory Control and Data Acquisition
SCAQMD	South Coast Air Quality Management District
SDI	Silt density index
sf	Square foot
SMACNA	Sheet Metal and Air Conditioning Contractor's National Association
SOR	Surface overflow rate
SRT	Solids retention time
TDH	Total dynamic head
TDS	Total dissolved solids
TKN	Total Kjeldahl Nitrogen
TN	Total nitrogen

TOC	Total organic carbon
TP	Total phosphorus
TS	Total solids
TSS	Total suspended solids
UL	Underwriters Laboratories
USEPA	United States Environmental Protection Agency
UPC	Uniform Plumbing Code
UV	Ultraviolet
VFD	Variable frequency drive
VOC	Volatile organic compound
VS	Volatile solids
VSS	Volatile suspended solids
VTSH	Vertical turbine solids handling
WAS	Waste activated sludge
WEF	Water Environment Federation
WVWRF	West Valley Water Reclamation Facility
WWTP	Wastewater treatment plant

EXECUTIVE SUMMARY

In accordance with the agreement for Engineering Services dated July 3, 2017, AECOM prepared this Preliminary Design Report (PDR) for The Mission Springs Water District West Valley Water Reclamation Facility (WVWRF). The proposed facility is a new wastewater treatment plant to be located in Desert Hot Springs, California. The main purpose of the WVWRF is to increase the capacity of wastewater treatment within the District with a view to the future.

The method of effluent disposal is land application by discharge to infiltration basins. A groundwater modeling study determined the proposed infiltration basins would be located a sufficient distance downgradient from the nearest public water supply well to not affect the water supply to the well.

The following is a summary of the design values selected for initial WVWRF:

Table ES-1 -Summary of Influent Design Values

Parameter	Units	Value
Flow, ADMM	mgd	1.5
BOD5	mg/L	330
TSS	mg/L	370
TKN	mg/L	60
Ammonia Nitrogen	mg/L	43

Table ES-2 -Summary of Effluent Design Values

Parameter	Units	Value
Flow, ADMM	mgd	1.5
BOD_5	mg/L	30
TSS	mg/L	30
Total Nitrogen, Annual Average	mg/L	10
pH	Std. Units	6.0 – 9.0
TDS	mg/L	400 mg/l above domestic source water

Based on the review of alternatives outlined in this PDR, the following facilities are recommended for the first phase of the WVWRF:

- Influent Pump Station
- Preliminary Treatment – Metering, screening, and grit removal
- Secondary Treatment - Sequential Batch Reactors with submerged mixers, diffused aeration, waste sludge pumping, and decant tanks
- Effluent Pumps
- Infiltration Basins
- Aerated Sludge Storage with Solids Thickening
- Solids Dewatering Building
- Odor Control
- Emergency Stand-by Power Generator
- Administration/Electrical Building

Provisions will be made for adding future facilities in anticipation of the MSWD advancing their goals to recycle water, including:

- Fine screens
- Membrane Bioreactors
- Return Pumping
- Coagulation/Filtration
- Disinfection

The estimated construction cost for the WVWRF is $27.4 million (based on current June, 2018, Engineering News Record – ENR 20 Cities Index of 11,000). The total estimated project cost is $31.6 million that includes a 40% allowance for contingency and project services. A 24-month construction period is recommended for WVWRF construction.

If a lower initial capital investment is determined to be in MSWD's best interest, it may be possible to start with a lesser Phase 1 treatment capacity and defer some of the new facilities to future improvements. See Final Value Engineering Technical Memorandum for details on feasible facility/equipment deferment options and their cost reduction potential. The proposed Phase 1 design flow of 1.5 mgd provides capacity that is beyond the ten-year planning horizon. By the end of Year 1, flows are projected to be 0.29 mgd. Flows are projected to gradually increase to 1.0 mgd by Year 7 and 1.2 mgd by Year 9.

The Phase 1 capacity of 1.5 mgd will be revisited during detailed design workshops to best match initial capacity with projected flows and financial planning.

Section 1– Introduction

On July 3, 2017, the Mission Springs Water District (MSWD) authorized AECOM to prepare a Preliminary Design Report (PDR) for The West Valley Water Reclamation Facility (WVWRF) construction. The plant is located in the southwest portion of the MSWD service area in Desert Hot Springs, California.

1.1 Purpose

The purpose of this PDR is to review and analyze background information and data to provide engineering recommendations for the planned WVWRF. The PDR includes findings and recommendations for the treatment processes to be used at the new WVWRF. The PDR summarizes pre-design work activities and critical decisions supporting the project into the final design phase. The PDR coincides with a preliminary design completion status of 30% or less which is intended to document the basis of planning for the California Environmental Quality Act (CEQA) documentation being prepared by others. After the required CEQA clearances and approvals, the project would be ready to move forward into detailed design. The alternatives considered are intended to meet effluent quality limits in compliance with anticipated Waste Discharge Requirements (WDR) to be issued by the Colorado River Basin Regional Water Quality Control Board (CRBRWQCB). The PDR also includes a preliminary opinion of probable construction cost for the recommended improvements.

1.2 Background Information

MSWD provides water and sewer services to the communities of Desert Hot Springs, West Garnet, North Palm Springs, and various portions of unincorporated Riverside County. MSWD currently has 7,200 sewer connections throughout its service area.

It is understood that MSWD is implementing its Groundwater Quality Protection Program (GQPP) with objectives to remove from service individual septic systems that overlie the Mission Creek and Desert Hot Springs groundwater sub-basins, collect and treat the wastewater, and ultimately replenish the Mission Creek sub-basin. The GWPP is intended to protect groundwater quality from degradation by discharges from septic drain fields. The GWPP would ultimately remove more than 7,200 septic tanks for connection to MSWD's sewer system. As the GWPP implementation progresses, it has created the need for additional sewage treatment capacity within the MSWD service area. A portion of this added sewage flow would be diverted to the planned MSWD WVWRF located along the west side of Little Morongo Road, between 19th Avenue and 20th Avenue. The proposed site consists of 60-acres of undeveloped land owned by MSWD.

Based on the Comprehensive Wastewater Facilities Strategic Plan by Tetra Tech, September 17, 2018, the WVWRF would be constructed in phases with ultimate "build-out" capacity of up to 20 million gallons per day (mgd). The first phase would have an initial maximum monthly average treatment capacity of 1.5 mgd. The WVWRF would be planned, designed, and implemented to permit MSWD to allow further expansion with minimal demolition and removal of any Phase 1 facilities.

Recent studies completed for MSWD that are relevant to the development of this report are referenced as follows:

- Preliminary Design Report for Horton Wastewater Treatment Plant Expansion No. 5, June 2007.
- Geotechnical Investigation Report, Proposed Future Regional Wastewater Facility, Desert Hot Springs, California, April 2008.

- Comprehensive Wastewater Facilities Strategic Plan, Mission Spring Water District, September 17, 2008.
- Preliminary Design Report, Interstate 10 and Indian Avenue Area Sewer System, Mission Springs Water District, July 2012.
- Mission Creek and Garnet Hill Water Management Plan, Groundwater Flow Model of the Mission Creek and Garnet Hill Sub-basins and Palm Springs Subarea, Riverside County, California, January 2013.
- Initial Study for Mission Springs Water District Well 33 Solar Photovoltaic Electric Generating System Project, October 2015.
- 2015 Urban Water Management Plan, Mission Springs Water District, June 20, 2016.
- Recycled Water Program Development Feasibility Study, Technical Memorandum No. 2, Mission Springs Water District, Draft August 11, 2017.
- Study to Evaluate the Effects of Nitrogen Discharges to Groundwater, Alan L. Horton Wastewater Treatment Plant Desert Hot Springs, California, August 4, 2017.
- Groundwater Model To Evaluate The Potential Impact From The Proposed West Valley Water Reclamation Facility Percolation Basins, EnviroLogic Resources, March 2018.

Effluent from the proposed WVWRF is planned to be discharged to on-site infiltration basins. This is consistent with MSWD's 2008 strategic planning report.

The results of groundwater flow and transport modeling by EnviroLogic Resources show that the discharge to the infiltration basins will not impact the nearby public water supply well (Well 33). After the findings of this study, MSWD and AECOM met with RWQCB to discuss the plant effluent quality requirement.

All planning for the proposed WVWRF is based on the reasonable assumption that the WVWRF will be allowed to use the same method of effluent disposal as used at the Horton WWTP and achieve the same level of treatment as achieved at the Horton WWTP with one exception of total nitrogen. In relation to the "Preliminary Design Report for Horton Wastewater Treatment Plant Expansion No. 5", MSWD previously held discussions with the CRBRWQCB about the possible upgrade and expansion to the Horton WWTP. At that time, the CRBRWQCB expressed concerns about possible negative impacts of nitrogen on the local groundwater quality. They further made it clear that in the absence of a study, any expansion to the Horton WWTP would trigger a new requirement to remove total nitrogen (TN) (through denitrification) to effluent levels below 10 mg/L.

Therefore, the new WVWRF was planned to be designed to achieve the same level of treatment as the Horton WWTP with additional denitrification process to achieve TN below 10 mg/l.

The CRBRWQCB's concerns about the impact on nitrate levels in groundwater near the Horton WWTP resulted in the "Study to Evaluate the Effects of Nitrogen Discharges to Groundwater, Alan L. Horton Wastewater Treatment Plant Desert Hot Springs, California, August 4, 2017." Water quality data obtained from groundwater monitoring wells were reviewed and found to be slightly greater than background levels; however, the nitrate concentrations are below the maximum contaminant level (MCL) of 10 mg/L for nitrate in drinking water set by the United States Environmental Protection Agency (EPA).

1.3 Current Regulatory and Effluent Permit Requirements

MSWD is committed to water recycling as a future consideration. MSWD is proposing a new WVWRF as a first step. Initially, the level of treatment will be secondary with denitrification discharging to onsite infiltration basins. Provisions will be made to accommodate upgrades to advanced secondary and tertiary treatment as future steps toward producing recycle water depending on growth, demand, and available funding.

The project is subject to the permitting process established by the California Environmental Quality Act (CEQA).

Other permits required for construction of the WVWRF may include:

- City of Desert Hot Springs;
- Riverside County "Plan Check" Department and Flood Control and Water Conservation District;
- California State Water Resources Control Board;
- Colorado River Basin Regional Water Quality Control Board (CRBRWQCB);
- Federal Emergency Management Agency (FEMA)/ US Army Corps of Engineers;
- California Department of Health (if recycled water distribution is included);
- South Coast Air Quality Management District (SCAQMD); and
- US Environmental Protection Agency.

The CRBRWQCB is the primary agency responsible for establishing effluent quality limits for discharges from municipal wastewater treatment plants. A summary of this agency's requirements is presented below with more details presented in the Permit Requirements Technical Memorandum.

MSWD is autonomous with regards to permits for building within MWSD property. Other permitting agencies that will likely be involved are the Riverside County Flood Control and Water Conservation District for review of stormwater management plans and South Coast Air Quality Management District for review of sources of air emissions including emergency generator engine exhaust and odors.

1.3.1 CRBRWQCB Waste Discharge Requirements

The proposed effluent discharge is land application to infiltration basins operated for the purpose of effluent disposal. There is no requirement for tertiary treatment for the initial 1.5 mgd treatment plant. Provisions will be made to accommodate a possible upgrade to tertiary treatment with the addition of coagulation/filtration and disinfection as may be needed to allow water recycling in the future.

The State of California defines a Groundwater Replenishment Reuse Project (GRRP) as "a project involving the planned use of recycled municipal wastewater that is operated for the purpose of replenishing a groundwater basin designated in the Water Quality Control Plan [as defined in Water Code section 13050(j)] for use as a source of municipal and domestic water supply."

The proposed project does not meet the definition of a GRRP because the proposed discharge is for land application intended for effluent disposal and the discharge is not intended for groundwater replenishment.

Preliminary meetings were held with the CRBRWQCB to discuss the project concepts. An application for Waste Discharge Requirements will be submitted to the CRBRWQCB and will consist of a Report of Waste Discharge (ROWD), including State Form 200, a technical report that thoroughly characterizes the discharge, a groundwater modeling report, and an antidegradation analysis.

On 08 May 2018, MSWD held a second meeting with the CRBRWQCB to discuss the proposed project. MSWD proposed the level of wastewater treatment to be secondary with denitrification before discharging to onsite infiltration basins. No disinfection is planned with the initial WWTP.

The presence of MSWD's potable water supply Well 33 located approximately one-half mile from the proposed Regional WWTP spreading basins raised questions regarding the potential degradation of the drinking water supply to Well 33. Groundwater modeling indicates that the infiltration basins are located downgradient in the direction of groundwater flow from Well 33 and will not impact the quality of water to Well 33.

Groundwater modeling used conservative “worst case” assumptions to provide an initial assessment of the potential impact that the proposed discharge would have on groundwater quality at the intake of Well 33. See Permit Requirements Technical Memorandum for more details regarding CRBRWQCB Waste Discharge Requirements.

Section 2 – Review of Existing Conditions

This section discusses the existing conditions at the WVWRF. This information would be considered during the preliminary engineering, design and construction of the improvements.

2.1 Climate

Desert Hot Springs has a hot, windy, desert climate. Data for Palm Springs is the closest weather recording station and is considered to have similar weather as Desert Hot Springs. Monthly average temperatures range from a high of 108.1 degrees Fahrenheit (°F) in July to a low of 65.0°F in January. Precipitation averages less than 5 inches per year. Additionally, there have been historically high levels of evaporation in the area. Monthly average evaporation ranges from 2.5 inches in the winter up to 15 inches in the summer. Table 2-1 summarizes climate data for Palm Springs.

Table 2-1 – Climate Data for Palm Springs Int'l Airport (1981–2010 normals)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Record high °F (°C)	95 (35)	99 (37)	104 (40)	112 (44)	116 (47)	121 (49)	123 (51)	123 (51)	121 (49)	116 (47)	102 (39)	93 (34)	123 (51)
Average high °F (°C)	65.0 (18.3)	73.9 (23.3)	80.5 (26.9)	87.5 (30.8)	95.6 (35.3)	103.6 (39.8)	108.1 (42.3)	107.3 (41.8)	101.7 (38.7)	91.1 (32.8)	78.4 (25.8)	66.0 (18.9)	88.23 (31.23)
Average low °F (°C)	35.0 (1.7)	48.0 (8.9)	52.2 (11.2)	57.4 (14.1)	64.4 (18)	71.0 (21.7)	77.6 (25.3)	77.6 (25.3)	71.7 (22.1)	62.5 (16.9)	50.0 (10)	44.2 (6.8)	59.3 (15.17)
Record low °F (°C)	19 (−7)	24 (−4)	29 (−2)	34 (1)	36 (2)	44 (7)	54 (12)	52 (11)	46 (8)	30 (−1)	23 (−5)	23 (−5)	19 (−7)
Average precipitation inches (mm)	1.15 (29.2)	1.11 (28.2)	0.53 (13.5)	0.06 (1.5)	0.02 (0.5)	0.02 (0.5)	0.13 (3.3)	0.29 (7.4)	0.23 (5.8)	0.24 (6.1)	0.32 (8.1)	0.87 (22.1)	4.97 (126.2)
Average precipitation days (≥ 0.01 in)	3.1	3.2	1.6	0.6	0.2	0	0.6	0.9	0.8	0.7	0.8	1.9	14.4

Source: NOAA

Table 2-2 provides monthly average pan evaporation at nearby observation stations. Indio Fire Station is considered to be most representative of Desert Hot Springs because of similar historic monthly average temperatures which are nearly identical to those recorded at Palm Springs Int'l Airport. Pan evaporation rates at the Indio Fire Station range from 2.5 inches in December to 15.0 inches in July, with an annual total of 105.4 inches.

Table 2-2 – Monthly Average Pan Evaporation at Nearby Observation Stations (inches)

Observation Station	Period of Record	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Beaumont Pumping Plant	1948-1975	2.9	3.3	4.1	5.0	6.4	8.2	10.6	10.0	7.9	5.9	3.2	2.9	70.4
Beaumont 1 E	1948-2001	3.1	3.7	5.0	5.2	7.6	9.3	11.0	10.7	8.9	6.5	5.2	4.0	80.1
Indio Fire Station	1927-2005	2.9	4.4	7.2	10.0	12.7	14.9	15.0	13.6	10.8	7.6	4.0	2.5	105.4

Not all of the water discharged to the proposed infiltration basins will be discharge to groundwater. A portion discharge to the proposed infiltration basins will be lost to evaporation from the surface of the standing water, and also from the upper layer of surficial soils (vadose zone) as the infiltration basins are allowed to rest and dry out. Calculations for the volume of water discharge to groundwater are recommended to include a correction to adjust for water lost to evaporation.

MSWD operations staff report that, in Desert Hot Springs, very strong sustained winds are common and are predominately from the west. Wind roses based on data from nearby meteorological stations show the predominant wind direction to be west northwest as shown in Figure 2-1.

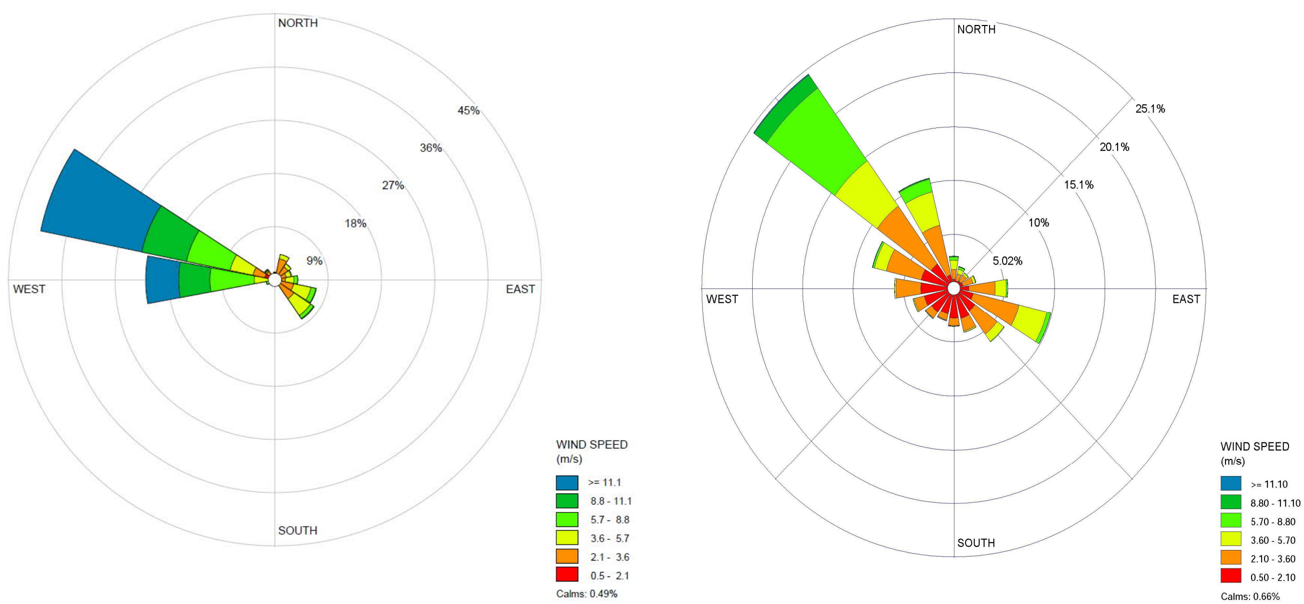


Figure 2-1 – Whitewater Wind Rose (2014-2016) (Left)**; Palm Springs Wind Rose (2012-2016)** (Right)

The design of new facilities will consider adding resilience against the impacts and damaging effects of strong winds and occasional sand storms on equipment and operations and maintenance activities.

2.2 Geology

The geology of Desert Hot Springs is alluvial material deposited over geologic fault lines. A branch of the San Andreas Fault aligns roughly east-west across the area, which splits the groundwater alluvial aquifer into multiple sub-basins. The proposed site lies within the Garnet Hill sub-basin and is on a gentle south sloping alluvial fan surface within the general influence of the Mission Creek watershed. A primary wash of the Mission Creek watershed lies approximately 0.15 miles to the east of the site. AECOM's geotechnical investigation did not encounter groundwater on the site. The borings were performed to a maximum depth of approximately 50 feet below ground surface (bgs).

More detailed geologic information is presented in Section 7.3 and the Geotechnical Investigation technical Memorandum.

2.3 Existing Wastewater Flows

Historically, flows to the Horton WWTP and Dos Palmas pump station were recorded weekly. MSWD started recording flow data digitally at 5-minute intervals beginning in April 2017. This 5-minute interval data was used to analyze the diurnal influent flow pattern. The Horton WWTP typical diurnal influent flow curve for a 24-hour cycle is presented in Figure 2-2.

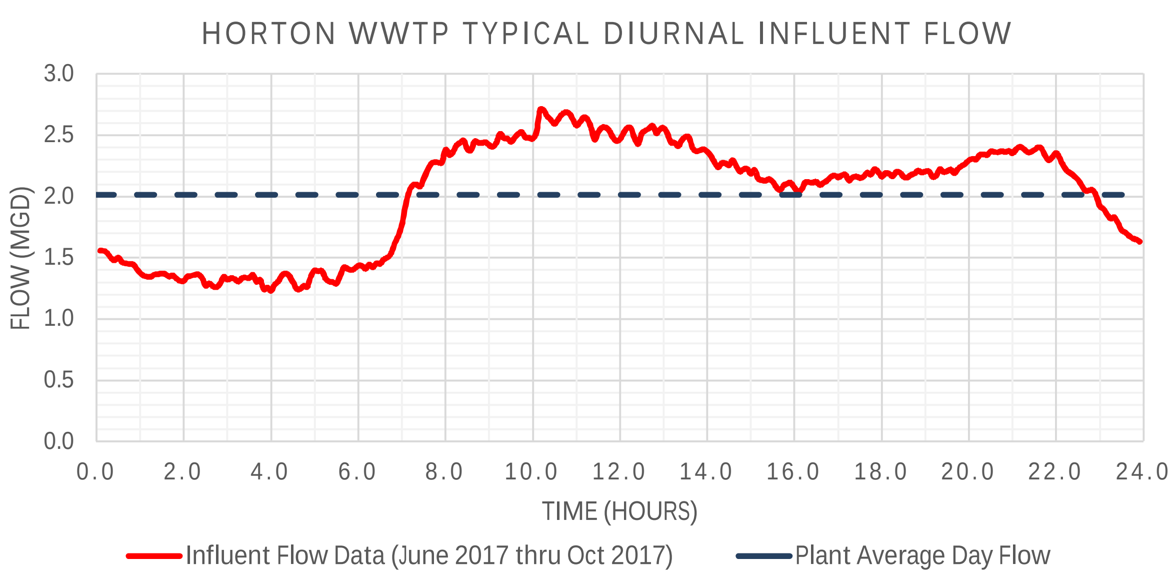


Figure 2-2 – Horton WWTP Typical Diurnal Flow

The curve was developed based on hourly influent flow data collected by plant operators from June 2017 to October 2017. The influent flow to the plant ranges from approximately 1.2 mgd at low flow to approximately 2.7 mgd at peak flow. The peak flow occurs between the hours of 10:00 AM and 12:00 PM. The daily average flow for the Horton plant was approximately 2.0 mgd.

MSWD provided additional flow data from the Horton plant that ranged from June 2017 through September 2018. Based on this data, the maximum 30-day rolling average is 2.0 mgd, the peak day flow is 2.2 mgd, and the peak hour flow is 3.6 mgd.

The Dos Palmas Lift Station typical diurnal influent flow curve for a 24-hour cycle is presented in Figure **2-3**.

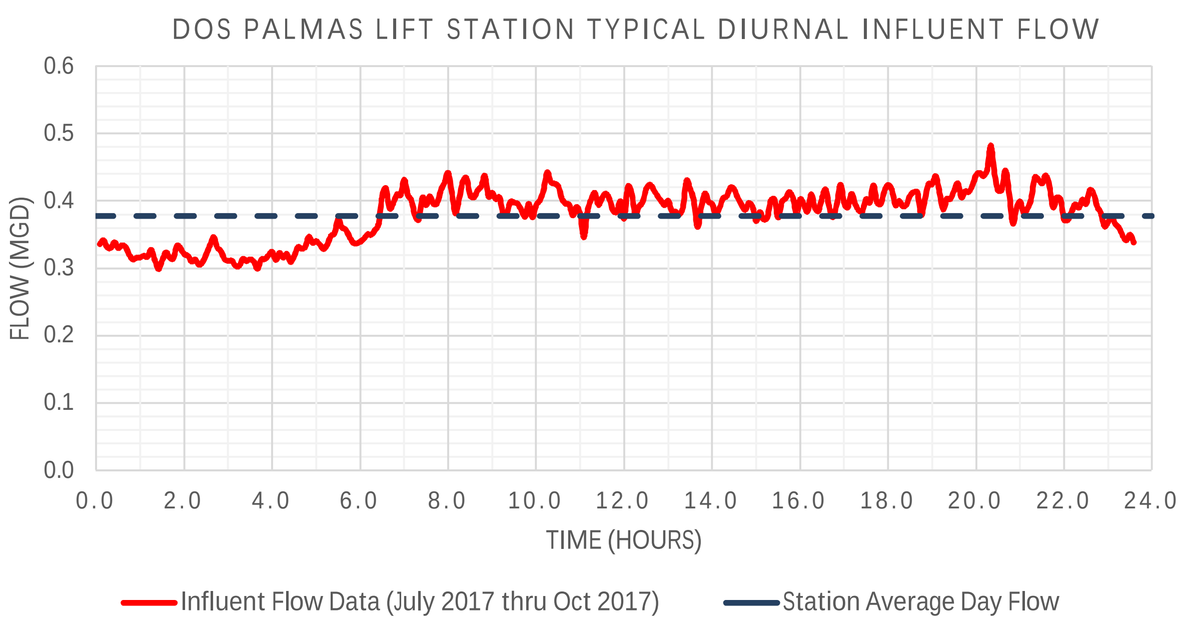


Figure 2-3 – Dos Palmas Lift Station Diurnal Flow

The curve was developed based on hourly influent flow data collected by plant operators from July 2017 to October 2017. The influent flow to the station ranges from approximately 0.30 mgd at low flow to approximately 0.48 mgd. The peak flow occurs between the hours of 8:00 PM and 10:00 PM. The daily average flow for the Dos Palmas station is approximately 0.38 mgd with an average daily peak hour flow of approximately 0.42 mgd.

2.4 Existing Wastewater Characteristics

Historical data from Horton WWTP shows fluctuating levels of influent BOD and TSS throughout the year.

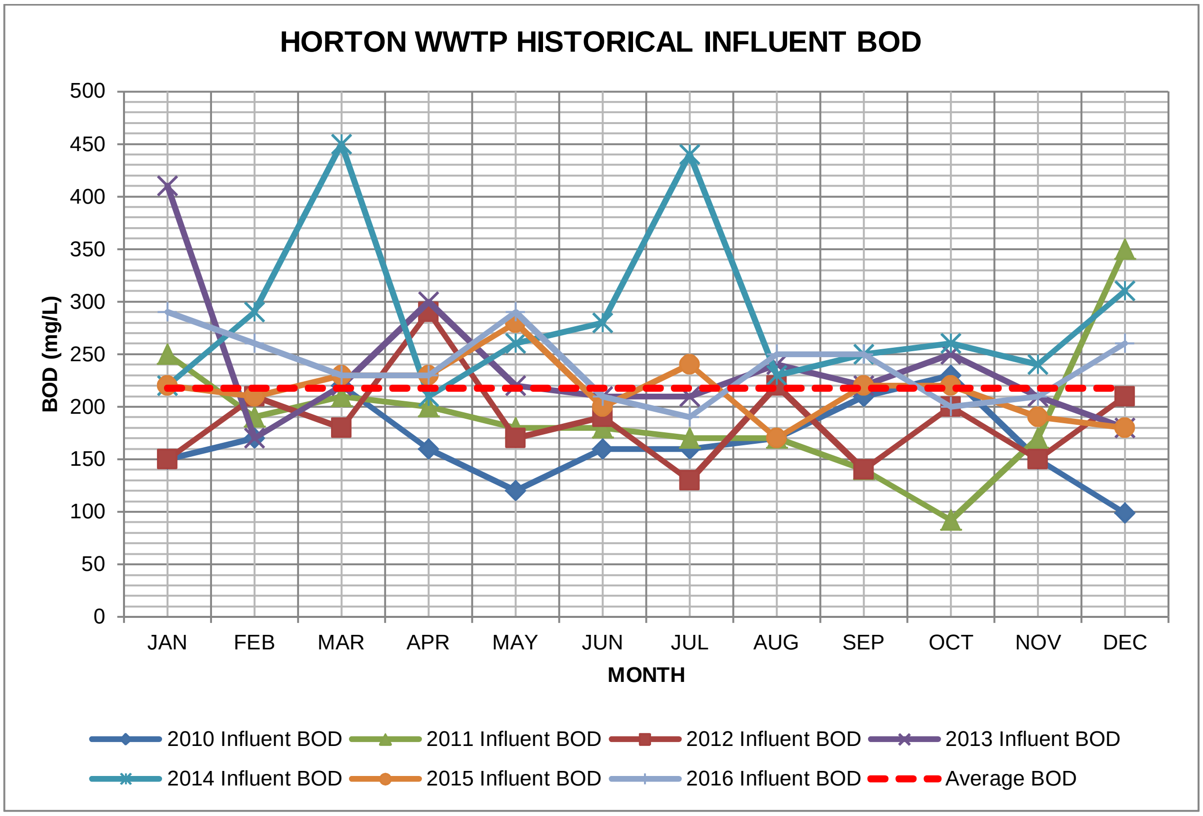


Figure 2-4. Influent BOD levels fluctuated greatly during some years, but generally BOD at Horton WWTP ranged between 150 to 275 mg/L. Average BOD was about 220 mg/L from 2010 through 2016.

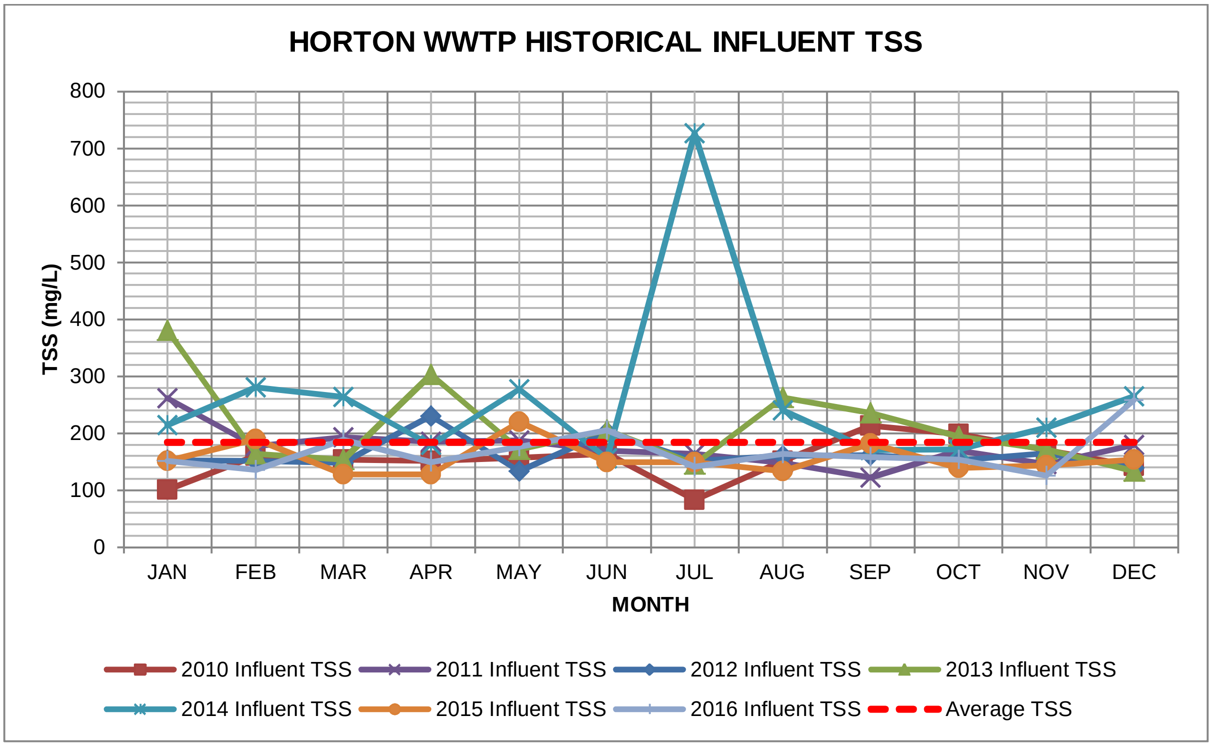


Figure 2-5. TSS levels ranged between 150 to 250 mg/L. Average TSS at Horton WWTP was about 185 mg/L from 2010 through 2016.

A large spike in influent BOD_5 and TSS was recorded in July 2014. Occasional spikes in the data may be attributed to side streams influencing the data because side streams are returned to the head of the plant at a location that is upstream of the sampling point.

2.5 Existing Energy Rates

Based on energy usage data from the Horton WWTP from January 2015 to July 2017, the electricity costs are $0.10 per kWh. Current natural gas and diesel fuel costs will be obtained for use during final design.

2.6 Evaluation of Existing Facilities and Impacts on WVWRF

MSWD currently operates two wastewater treatment plants. The Horton Plant is a 2.3 mgd facility that receives the majority of the total flow. The Desert Crest Plant is a 0.18 mgd package-type unit intended to primarily serve the Desert Crest development located east of main sewer service area serving Desert Hot Springs.

2.6.1 Horton WWTP

The MSWD Horton WWTP is currently designed for the following flows and loads:

- Average flow 2.3 mgd
- Peak hourly flow 5.0 mgd

- Influent BOD_5 4,800 lb/day (250 mg/L)
- Influent TSS 4,800 lb/day (250 mg/L)

The HWWTP discharges treated effluent through a 24-inch outfall into a series of five infiltration basins. The current effluent permit limitations are listed in Section 1.3.

The HWWTP is currently permitted to discharge treated effluent to infiltration basins. Concentrations of biochemical oxygen demand (BOD_5), total suspended solids (TSS), and total dissolved solids (TDS) in the treated effluent discharged to the infiltration basins should not exceed the limits listed in Table 2-3.

Table 2-3 – Current Effluent Requirements for Horton WWTP

Parameter	Monthly Average	Weekly Average
Flow	2.3 mgd	---
BOD_5	30 mg/L (500 lb/day)	45 mg/L (750 lb/day)
TSS	30 mg/L (500 lb/day)	45 mg/L (750 lb/day)
TDS	(Note 1)	---
pH	6 to 9	---
Note 1: TDS must not exceed the domestic water supply by more than 400 mg/L.		

A PDR was prepared in June 2007 to upgrade and expand the Horton WWTP. The PDR summarizes the unit processes and the existing facility, develops and presents design criteria, and evaluates and recommends treatment options for the upgrade and expansion. At that time, MSWD elected to proceed with some of the recommended upgrades and to defer the decision to expand the plant capacity.

The permitted capacity was increased from 2.0 to 2.3 mgd in 2014 as a result of some of the interim improvements from Phase 5 design. During the permit renewal process, the CRBRWQCB raised concerns about the impact of the discharge on groundwater nitrogen concentrations. They indicated likelihood that increasing Horton permitted flows greater than 2.3 mgd would trigger an effluent limit on Total Nitrogen (TN) of 10 mg/L which would require an upgrade to the treatment process to accomplish denitrification.

2.6.2 Desert Crest WWTP

The Desert Crest WWTP is a 0.18 mgd package-type unit intended to primarily serve the Desert Crest development located east of main sewer service area serving Desert Hot Springs.

The Desert Crest WWTP has a WDR permitted capacity of 0.18 mgd. Current flows are estimated to be 50,000 gpd. Long-range plans are to take Desert Crest WWTP offline and deliver flows to the proposed WVWRF.

2.6.3 Collection and Conveyance Facilities

A new wastewater conveyance system is planned to deliver wastewater from the City to the proposed WVWRF. A gravity trunk sewer is planned along Little Morongo Road to deliver wastewater from the north. The discharge from the Dos Palmas pump station will be redirected to the proposed Little Morongo Road trunk sewer. A new influent pump station is proposed at the WVWRF site to receive incoming flows.

A new gravity line has been recently proposed by cultivation businesses located north and west of the site.

A new pressure force main is planned to deliver wastewater from the I-10 corridor. The Interstate 10 and Indian Avenue Area Sewer System Preliminary Design Report identified a proposed pressure force main to service the planned commercial development along the I-10 corridor.

2.7 Existing Residuals Handling and Disposal Practices

Existing residuals (waste biosolids, sludge) from the Horton Plant are dewatered on site. Dewatered sludge is hauled off-site by a contract hauler for beneficial use at a composting facility.

Section 3 – Design Conditions

3.1 Flows

The Phase 1 design flow for the proposed facility is 1.5 mgd with an initial flow of 0.2 mgd on the first day of plant operations. The 1.5 mgd facility will be designed to accommodate a future expansion to 3.0 mgd. The ultimate capacity of the regional plant site is 20 mgd. Peaking factors for Max Day and Peak Hour based on flow data from the Horton Plant from July 2017 through September 2018 are 1.1 and 1.8, respectively. Peaking factors based on textbooks and engineering best practices of1.5 and 2.5 for Max Day and Peak Hour, respectively, may be used for final design.

The initial flow of 0.2 mgd is planned to be diverted from the Horton Plant by redirecting the discharge from the Dos Palmas pump station located at Dillon and Bubbling Springs Road, west of Avenida Manzana. The remainder of the flow to the WVWRF will be from "new" sources consisting of septic tank conversion projects and new residential and commercial development.

MSWD provided a copy of the 2017 Regional Wastewater Program Flow Projections Technical Memorandum, prepared by TKE Engineering, which provided updated estimates of projected flows for the next 20 years. These flow estimates were based on MSWD's continued success of its septic to sewer conversion program (GQPP), development trends, and land use changes on the first day of operation at the WVWRF, the monthly annual average flow is expected to be 0.23 mgd which is the flow diverted from the existing Dos Palmas lift station. By the end of Year 1, flows are projected to be 0.29 mgd assuming plans to convert septic tanks to central sewer (0.04 mgd) and plans to connect I-10 corridor (0.05 mgd) are realized. Monthly average annual flows are projected to gradually increase to 1.0 mgd by Year 7 and 1.2 mgd by Year 9 as a result of continued GQPP implementation and new service connections from growth. The selected Phase 1 capacity of 1.5 mgd would take the plant capacity beyond the flow projections for Year 9. The capacity of 1.5 mgd is a firm, reliable capacity that will have redundant backup built in. The SBR tanks will be able to treat 1.5 mgd with one unit out of service and will have enough equalization volume to handle peak hour flows.

Table 3-1 summarizes wastewater influent design flows.

Table 3-1 – Wastewater Influent Design Flows

Parameter	Unit	Initial Flow (Day 1)	Phase 1 Design Capacity	Future Design Capacity	Ultimate Capacity
Avg Daily Max Month	mgd	0.20	1.50	3.0	20.0
Max Day	mgd	0.30	2.25	4.5	---
Peak Hour	mgd	0.50	3.75	7.5	---

3.2 Pollutant Loads

The concentrations of influent flow constituents to the proposed WVWRF is an estimate based on available data to the Horton WWTP, combined with a forecast of future flows from new sources based on textbook data for estimating pollutant loadings per capita. Max Month and Peak Day loadings are based on assumed peak factors of 1.1 and 1.4 respectively.

Table 3-2 summarizes wastewater influent characteristics. Table 3-3 summarizes wastewater influent design concentrations and loadings.

Table 3-2 – Wastewater Influent Characteristics

Parameter	Unit	Average Day	Source
BOD	mg/l	330	Horton Plant #5 Expansion and M&E textbook values
TSS	mg/l	370	Horton Plant #5 Expansion and M&E textbook values
VSS	mg/L	330	Horton Plant Sampling Results TSS/VSS Ratio
TKN	mg/L	60	Horton Plant #5 Expansion and M&E textbook values
NH4-N	mg/L	43	Per Sampling NH_4-N/TKN
TP	mg/L	10	Horton Plant #5 Expansion and M&E textbook values
Alkalinity	mg/L	200	Conservative Concentration assumed
Temperature, max	Deg C	30	Horton Plant data
Temperature, min	Deg C	22	Horton Plant data

Table 3-3 – Wastewater Influent Design Concentrations and Loadings

Parameter		Unit	Initial Flow (Day 1)	Plant Design Capacity
BOD	Concentration	mg/L	330	330
	Average Annual Load [1]	lb/d	550	4,128
	Average Day Max Month [2]	lb/d	605	4,541
	Maximum Day Load [3]	lb/d	770	5,779
TSS	Concentration	mg/L	370	370
	Average Annual Load [1]	lb/d	617	4,629
	Average Day Max Month [2]	lb/d	679	5,092
	Maximum Day Load [3]	lb/d	864	6,481
VSS	Concentration	mg/L	330	330
	Average Annual Load [1]	lb/d	550	4,128
	Average Day Max Month [2]	lb/d	605	4,541
	Maximum Day Load [3]	lb/d	770	5,779
TKN	Concentration	mg/L	60	60
	Average Annual Load [1]	lb/d	100	751
	Average Day Max Month [2]	lb/d	110	826
	Maximum Day Load [3]	lb/d	140	1,051
NH_4-N	Concentration	mg/L	43	43
	Average Annual Load [1]	lb/d	72	538

Parameter		Unit	Initial Flow (Day 1)	Plant Design Capacity
	Average Day Max Month [2]	lb/d	79	592
	Maximum Day Load [3]	lb/d	101	753
TP	Concentration	mg/L	10.0	10.0
	Average Annual Load [1]	lb/d	17	125
	Average Day Max Month [2]	lb/d	19	138
	Maximum Day Load [3]	lb/d	24	175
Alkalinity	Concentration	mg/L	200	200
	Average Annual Load [1]	lb/d	334	2,502
	Average Day Max Month [2]	lb/d	367	2,752
	Maximum Day Load [3]	lb/d	468	3,503

(1) Average Annual = Average for a 365 consecutive day period
(2) Average Day Maximum Month = Highest 28-day running average flow
(3) Maximum Day = Highest observed daily flow

3.3 Effluent Discharge Characteristics

Per Sections 1.2 and 1.3 and previous discussions with CRBRWQCB, the proposed target effluent quality criteria for WVWRF are summarized in Table 3-4. These effluent design criteria are consistent with the HWWTP WDR with the exception of TN. Based on previous talks with the CRBRWQCB, it is understood that the CRBRWQCB intends to add a discharge limit on TN in the event MSWD elects to increase the capacity of the HWWTP.

Table 3-4 – WVWRF Effluent Limits

Parameter	Unit	30-Day Arithmetic Mean	45-Day Arithmetic Mean	Annual Average
BOD	mg/L	30	30	-
TSS	mg/L	30	30	-
TN	mg/L	-	-	10
TDS	mg/L	400 mg/L above the domestic source water		-
pH	mg/L	6.0 - 9.0		-

In addition to the design criteria presented in Table 3-4, the infiltration basins shall be maintained so they will be kept in aerobic conditions. The dissolved oxygen content in the upper zone (one foot) of the evaporation/percolation ponds shall not be less than 1.0 mg/L.

3.4 Process Modeling

3.5 Solids Production Estimates

A review of available HWWTP influent wastewater characterization data found historical data for BOD and TSS. Wastewater characterization sampling is not recommended as it is not economically feasible for a treatment plant the size of the WVWRF. The data may also not be representative of the actual WVWRF influent characteristics. Biowin modeling will be performed for the WVWRF and will be based on calculated and estimated values. Solids Production Estimates

Waste biosolids (sludge) will be generated from the proposed WWTP. Quantities and percent solids concentrations will depend on the flows and the selected treatment process.

For the SBR process, at an initial flow of 0.2 mgd, the plant will generate 435 lb/day solids or less than 7,000 gpd of waste activated sludge at 0.75% solids concentration. At 1.5 mgd, the sludge generation will be proportionally greater at 3,500 lb/day solids or 55,000 gpd at 0.75% solids.

Section 4 – Projected Future Design Conditions

MSWD has indicated plans for a future recycled water program. At the time when the District is prepared to begin water recycling, the effluent discharge requirements for water recycling will need to meet Title 22 water recycle criteria for the intended use.

Table 4-1 summarizes the regulatory requirements for effluent quality in effect at the time of this report which are used as the basis for future planning. This assumes land application of municipal wastewater with the purpose of replenishing a groundwater basin. Effluent quality requirements for other reuse alternatives, such as golf course irrigation, may be allowed with less stringent treatment requirements.

Table 4-1 – Effluent Quality Characteristics for Groundwater Replenishment Reuse Project

Parameter	Unit		Limit	Comment
BOD	mg/L		30	-
TSS	mg/L		30	-
TN	mg/L		10	-
Turbidity	NTU	Disc Filter	2	24-hour average
			5	More than 5 % of the time within a 24-hour period
			10	At anytime
		Membrane Filter	0.2	More than 5 % of the time within a 24-hour period
			<0.5	At anytime
Total Coliform Bacteria	MPN/100 ml		<2.2	7 day average
			<23	No more than one sample in any 30 day period
			<240	At anytime
TOC/RWC	mg/L		≤0.5	Total Organic Carbon / Recycled Water Contribution

AECOM prepared conceptual designs and sizing for installing coagulation, filtration, and disinfection for both 1.5 mgd and 3 mgd to meet the Title 22 Recycled Water requirements at the WVWRF. This information was used as the basis for reserving space for future upgrades. A similar exercise was completed to approximate the size of facilities needed to accommodate ultimate buildout of the treatment plant site to 20 mgd. Details supporting the planning-level design concepts for Recycle Water and ultimate build-out of the treatment plant site are beyond the scope of this report and are not included as part of this document.

Section 5 – Review of Alternatives

Various treatment processes were evaluated to select the most appropriate methods and technologies to achieve the intended results. The proposed processes include:

- Influent pumping;
- Preliminary treatment;
- Biological treatment;
- Solids thickening;
- Solids dewatering;
- Solids stabilization; and
- Odor control system.

Some unit process selections required a life cycle comparison for a recommended selection. For the review of life cycle cost, the following unit costs and criteria were used:

- 20-year life cycle;
- 4% interest;
- Polymer cost = $1.15/pound;
- Sodium hydroxide cost = $3.00/gallon;
- Citric acid cost = $6.50/gallon;
- Parts and materials 1% of capital cost;
- Staff labor at $25 per hour plus fringe benefits at 40% of raw salary;
- Electric use based on 80% of motor nameplate ratings; and
- Energy costs are presented in Section2-5.

5.1 Influent Pumping

The influent pump station is required to hydraulically lift the wastewater from the collection system to the headworks of the treatment plant. The influent pumps are sized to accommodate the peak hour flow observed within the collection system and are operated in a lead/lag fashion. The influent pumps provide enough hydraulic head to allow the wastewater stream to gravity flow through the headworks and into the SBR tanks. The influent pumps must have a good solids handling ability and be able to operate continuously in a corrosive environment.

5.1.1 Option 1 – Submersible Pumps

The pumps would be located in a wet well. No dry well is required. Each pump would be provided with a guiderail system that would allow the pump to be raised to grade level by a lifting chain. The pumping equipment would work under submersible conditions. This option offers a lower construction cost than other options because it eliminates a dry well. Variable frequency drives (VFDs) and motor control centers (MCCs) would be located in the new electrical building room at the WVWRF. Backup power to the pumps would be provided by the new centralized standby generator located outside of electrical room.

Submersible wastewater pumps have a number of advantages. A major one is low initial cost. Because only one pit is necessary, it reduces initial investment. There is no need for ventilation, lighting or other equipment, which is normal for dry pits. Flooding problems are also eliminated. Another advantage is low operating cost. Submersible pumps have safety and noise reduction benefits, too, since the pumps are well below grade level. There is less chance for accidents from an exposed motor, and there is minimal noise when the pump is operating.

There is limited above grade equipment - only the MCC structure and a lifting davit for removal of the pumps would be above ground. Submersible pumps have high reliability and long life. It runs only when needed, reducing wear and electrical costs. Suction pipe clogging and net positive suction head (NPSH) problems are eliminated. The wastewater cools the motor naturally, which can lead to a longer life.

A disadvantage of submersible pumps is lower operating efficiency. Also, the pump and motor motors assembly must be lifted out of the wet well for maintenance repairs. A lifting device is needed.

5.1.2 Option 2 – Vertical Turbine Solids Handling Pumps

The vertical turbine motors would be located outdoors and above grade. Each pump's column housing, shaft, impeller, and suction would extend into the wet well. VFDs and MCCs would be located in the new electrical building room at the WVWRF. Backup power to the pumps would be provided by the new centralized standby generator located in the new electrical room. The motors and discharges are located above grade allowing easy access for preventative maintenance. The pumps have higher pump efficiency than standard centrifugal pumps and submersible pumps. Higher pump efficiencies result in lower annual electrical costs. This pumping arrangement provides robust pumping equipment and is readily expandable to accommodate future flows. The pumps would operate under flooded conditions.

Vertical Turbine Solids Handling pumps are very costly, and best used in unique applications. They are limited to capacities greater than 1,000 gpm. The pump's long shaft and intermediate bearings make maintenance difficult. A crane is required to remove the equipment for maintenance or repairs. These pumps can be noisier than submersible pumps because the motors are outdoors above grade.

5.1.3 Option 3 – Suction Lift Pumps

For suction lift pumping, the pump, motors, and discharge piping are located above grade allowing easy access for preventative maintenance. The motors would be enclosed in engineered enclosures to contain noise. The capacity of suction lift pump stations would be limited by the NPSH and specific speed requirements as stated on the manufacturer's pump curve under the most severe operating conditions.

All suction lift pumps would be provided with an air relief line on the pump discharge piping. This line would be located at the maximum elevation between the pump discharge flange and the discharge check valve to allow bleed-off of entrapped air. Air relief piping would have a minimum diameter adequate to purge air during priming. A separate air relief line would be provided for each pump discharge. The air relief line would terminate in the wetwell, or suitable sump that is open to the atmosphere.

All pumps, connections, shut-off valves, and check valves would be located in a separate vault either above or outside of the wetwell, allowing accessibility to both the wet well and pump/ valve vault for inspection and maintenance.

The pumps and motors would be located outdoors and above grade. VFDs and MCCs would be located in the new electrical building room at the WVWRF. Backup power to the pumps would be provided by the new centralized standby generator located outside the new electrical room.

Self-priming systems are difficult to maintain and need additional training for maintenance. Since pumps and motors would be located above grade, an enclosure to minimize noise would be required. These pumps are not as efficient as pumps with the wet end in the wetwell. Another disadvantage of a self-priming pump is that its proprietary design prevents competitive bidding.

5.1.4 Evaluation and Recommendation

It is recommended to implement submersible pumps for the influent pump station. Submersible pumps are very reliable solids handling pumps with the ability to pass a wide range of debris, including fibrous material and large diameter solids, without clogging or damaging the pump. Furthermore, the submersible nature of the pumps allows for a smaller wet well footprint which results in lower capital costs. No extensive measures are required to access the pumps, provide for noise attenuation, or for safety precautions. MSWD prefers same manufacturer and model of submersible pumps as the existing pumps at Horton WWTP.

5.2 Preliminary Treatment

Preliminary treatment is required to remove inert materials from raw wastewater to minimize damage to downstream equipment. Inert materials can clog pumps, pipes, and tanks, as well as have a detrimental effect on aerators and diffusers. Reduction of materials such as grit and large debris is necessary to protect downstream equipment and processes, reducing operation and maintenance costs.

5.2.1 Channel Mounted Grinders

Channel mounted grinders macerate both organic and inorganic materials. These units do not reduce inert materials from the liquid stream. Channel mounted grinders are currently used at the Horton WWTP. Due to the sensitivity of proposed downstream processes to debris, these are not considered a viable option for preliminary treatment.

5.2.2 Screening

Different levels of in-channel screening were considered. For the purpose of this report, coarse screens have ¼-inch openings and fine screens have 1 to 2 mm openings. Coarse screens are intended to remove large objects from the influent. Fine screens would protect sensitive downstream treatment processes that require a higher level of pre-treatment. Fine screens are typically used with in conjunction with coarse screens to protect downstream membrane bioreactors. Screens are typically fabricated entirely of stainless steel to resist the corrosive effects of gaseous hydrogen sulfide.

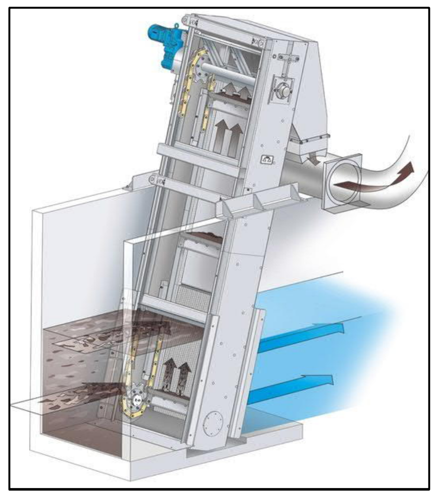


Figure 5-1 – Multi-rake Screen

5.2.2.1 Coarse Screen

Coarse screens remove sticks, rags, and other debris via vertical bars cleaned with a mechanical system. A mechanical system allows for removal of screenings at short intervals at high flow capacity. Brushes and wash water are sometimes required for this technology, as the rakes scrape debris from the bars and deposit them near the top of the apparatus for disposal. A ¼-inch coarse screen would remove solids such as sticks, rags, and other debris. Different types of commercially available coarse screens include

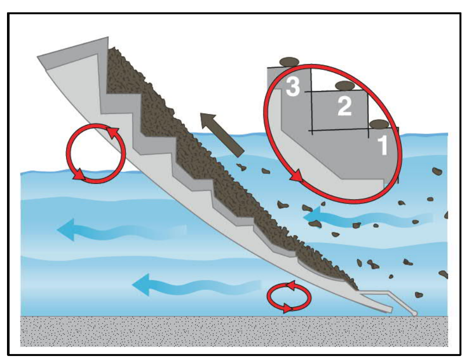


Figure 5-2 – Step Screen

multi-rake bar screens and step screens as shown in Figure 5-1 and Figure 5-2, respectively.

5.2.2.2 Fine Screen

Fine screens with openings of 2 mm or less are primarily installed to protect sensitive downstream process equipment. Different configurations of fine screens are commercially available including rotary drums, as shown in Figure 5-3, and band screens that use perforated plates or fine mesh screens. Screenings would be washed and dewatered before being hauled off for disposal. Screens are self-cleaning using wash water and/or brushes to remove accumulated solids from the screens.

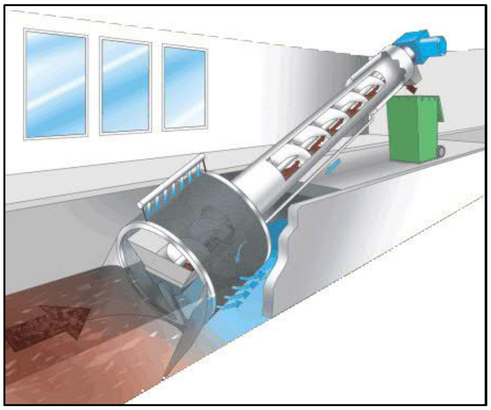


Figure 5-3 – Rotary Drum Screen

5.2.3 Grit Removal

The grit removal process will capture inorganic settleable solids such as sand, egg shells, coffee grounds and other inert material that can adversely impact downstream treatment processes through abrasion of mechanical equipment and accumulation in channels and process tanks.

Grit removal is a physical separation process whereby grit is settled out of the wastewater, then pumped to a classifier to be washed and dewatered for offsite disposal. Two options were considered for grit removal: vortex grit removal and Head Cell grit removal.

5.2.3.1 Vortex Grit Removal

Vortex grit removal uses the momentum of wastewater and vortex forces around the circular chamber to allow the grit to settle out and separate from the wastewater. The vortex grit chamber has a mechanical paddle that assists the vortex movement of wastewater and enhances the grit removal process. A cut-away view of a vortex grit unit is shown in Figure 5-4.

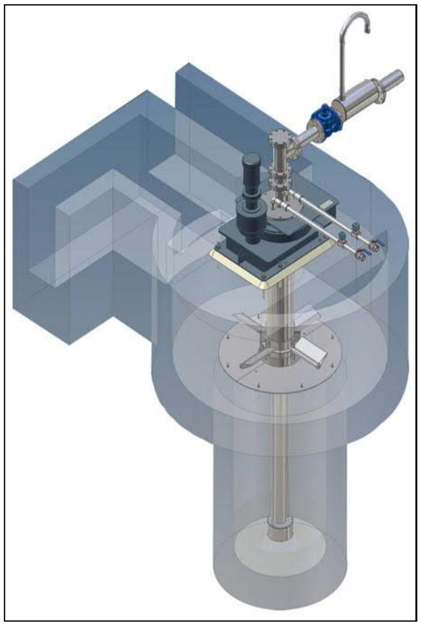


Figure 5-4 – Vortex Grit Removal

Wastewater enters the vortex grit chamber tangentially, flows around the 270 degree chamber and then exits the chamber parallel to the inlet. During this process, grit is settled out into the bottom of the chamber in a lower hopper, and is then pumped out periodically for washing, classification and dewatering. The grit pumps are recessed impeller pumps located below grade.

This type of grit removal process can achieve 95% removal of grit particles down to 106 microns.

Two vortex grit chambers will be provided; each designed to handle peak hour flow of 3.75 mgd. This configuration provides full redundancy if one chamber is taken offline.

An important design consideration for vortex grit removal is having a long, straight channel entering the grit removal chamber to ensure the wastewater flow becomes laminar, causing the grit to settle instead of being agitated through turbulence.

5.2.3.2 Head Cell Grit Removal

HeadCell is an emerging technology aimed at decreasing the overall footprint of the grit removal process while achieving the same removal efficiency. The HeadCell grit removal system works on the same principles as a vortex grit chamber, however a multiple tray separator increases the surface area of the chamber and provides shorter settling distances, which allows for more flow into the unit leading to an overall reduction in footprint. A cut-away view of a HeadCell grit removal unit is shown in Figure 5-5.

At the inlet of the HeadCell grit removal unit, an influent duct directs flow into a distribution header which evenly distributes the wastewater into the multi-tray system. Tangential feed into the HeadCell unit established a vortex flow pattern causing solids to fall into a boundary layer on each tray. Grit settles out by gravity along the sloped surface of each tray and is directed into a common collection sump in the center of the unit.

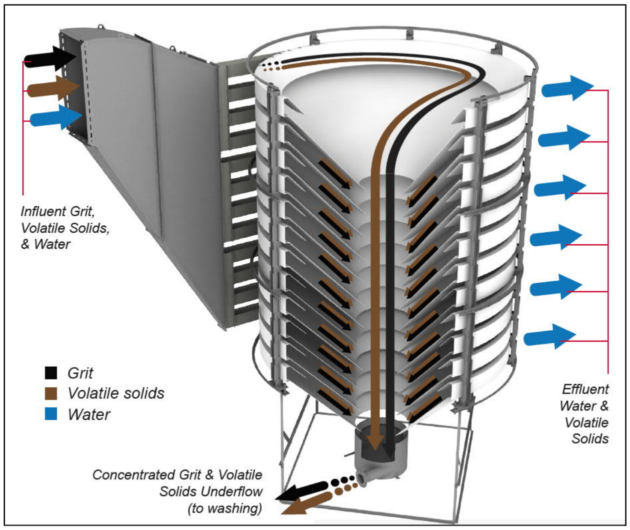


Figure 5-5 – HeadCell Grit Removal

Grit is pumped from the central collection sump to a grit classifier for washing and dewatering, and then sent for offsite disposal.

This type of grit removal process can achieve 95% removal of grit particles up to 106 microns. Two HeadCell units would be provided; each designed to handle peak hour flow of 3.75 mgd, to achieve full redundancy in the process.

This unit will also require a long, straight channel upstream to achieve laminar flow for more efficient grit settling.

5.2.4 Evaluation and Recommendation

Preliminary treatment is needed to prevent solids and debris from damaging downstream equipment and treatment processes. Coarse screening technologies are compared in Table 5-1. A multi-rake bar screen is selected for the basis of design based on durability and proven reliability. Fine screens are not required for the selected treatment process and were not evaluated in this report. However, lengths of open channels in the Headworks area will be reserved to add fine screens in the future.

Table 5-1 – Comparison of Coarse (1/4-inch) Screening Technologies

Technology	Opening Size	Advantages	Disadvantages
Multi-Rake Screen	¼-inch	• Simple design, low maintenance • Well established technology • Can be pivoted out of channel for maintenance • Low installed cost	• Requires downstream hydraulic control to provide submergence

Technology	Opening Size	Advantages	Disadvantages
Step Screen	¼-inch	• Self-cleaning; no brush or wash water • High capture rate with filter mat • No submerged sprocket or bearings	• Plates can dent if heavy material is encountered • More moving parts • Requires downstream hydraulic control to provide submergence

The channel mounted grinder was not considered due to sensitivity of downstream processes to particle size. A multi-rake style coarse screen was selected due to it being a well-established technology. If microfiltration (MF) or ultrafiltration (UF) systems are considered for the plant, fine screens will also be required. A perforated rotary drum was selected for its high capture rate and no need for downstream hydraulic control.

Vortex and HeadCell grit removal technologies are compared in Table 5-2.

Table 5-2 – Comparison of Grit Removal Technologies

Technology	Advantages	Disadvantages
Vortex Grit Removal	• Familiar, used at Horton WWTP • Established technology • Lower head loss • Lower installed cost	• Larger footprint • Lower grit capture
HeadCell	• Smaller footprint • Higher grit capture	• Higher head loss • Higher installed cost

Vortex grit chambers in duty/standby configuration are recommended for grit removal. Grit pumps will be recessed impeller pumps. Grit will be washed and dewatered in a grit classifier and grit disposed offsite.

Vortex grit removal is a well-established and reliable technology for small to medium sized plants. This technology has a relatively small footprint and high capture efficiency. Installation of vortex grit chambers will also enable the plant to retrofit HeadCell units in the future if desired. Retrofitting HeadCell units will increase capture efficiency of the process and allow more flow through the units as the plant expands.

As the basis of design, the following methods of preliminary treatment are recommended:

- Multi-rake ¼-inch bar screen coarse screen;
- Rotary drum fine screen (reserve channel space if needed in the future); and
- Vortex grit removal (with the potential to retrofit HeadCell units in the future if desired).

Dedicated screenings treatment equipment, such as washers and compactors, may be deferred until flows increase.

5.3 Primary Treatment

Primary clarifiers achieve primary treatment by sedimentation and removal of settleable solids to effectively reduce the loading on the secondary biological treatment system. Primary clarifiers also remove floating materials and scum from the wastewater. For relatively small flows like WVWRF Phase 1 (1.5 mgd) and future flow (3 mgd), primary clarifiers are not typically used. Primary clarifiers are not being considered in the initial phases for this project. Primary clarifiers may be warranted to accommodate higher flows in the future.

Primary cloth disc filters may be worthy of consideration as an alternate method of primary treatment. Cloth disc filters are an emerging primary treatment technology that has the potential to perform better, cost less than primary clarifiers, and could potentially eliminate the need for screening.

5.4 Biological Treatment System

Biological treatment processes were evaluated to meet initial treatment requirements and to be compatible with future plans to produce recycled water. Several biological alternatives were discussed during workshops with MSWD and two alternatives were selected for further evaluation: 1) Conventional Activated Sludge (CAS) using Sequencing Batch Reactor (SBR), and 2) Membrane Bioreactor (MBR).

MBR was initially identified in the most recent sewer master plan as the selected process to meet the goal of producing recycle water. However, the MSWD's preference is to defer water recycling to the future. SBR was selected as the lower-cost CAS alternative that better meets the initial effluent requirements. The SBR process is also better-suited to accommodate future upgrades to produce recycle water by either combining with disc filtration or converting to MBR.

5.4.1 Sequencing Batch Reactors

An SBR is an activated sludge process that operates in a fill and draw sequence. Whereas a traditional activated sludge process conveys wastewater through an anoxic tank, an aeration tank, and a clarifier, a sequencing batch reactor achieves the same processes in a single tank by altering the conditions to create an anoxic phase, an aerobic phase, a settling phase, and a decant phase during the treatment cycle. This approach reduces the footprint requirement considerably, as compared to conventional biological treatment and eliminates the need for return activated sludge (RAS) and nitrified recycle piping.

An SBR is a lower-cost alternative that can be readily converted to MBR. The batch treatment affords operational flexibility to meet effluent goals for disposal to infiltration basins. The process will have ability to accommodate the low initial flows.

To upgrade to produce recycle water, the filtration and disinfection can be added to the SBR process. SBR tanks can also be converted to MBR as a way to increase treatment capacity while also improving performance using the same tanks as the SBR process.

A total SBR tank volume of 1.8 mgal will be provided. This will be split into four 30 foot wide by 83 foot long by 21 foot deep basins. To handle the initial low flow conditions, one basin will be divided in half and operated as two smaller SBRs.

A total of 180,000 gallons of decant equalization is required to buffer peak flows at 1.5 mgd. The initial aeration demand of the process is 1,150 pounds of oxygen per day at 0.2 mgd, increasing to 8,600 pounds of oxygen per day at 1.5 mgd. Diffused air will be delivered from screw-lobe positive displacement blowers as they are able to handle pressure fluctuations well, which are expected in the SBR tanks. The use of turbocompressors with magnetic bearing technology is not ideal as they are not as effective or efficient running under fluctuating pressure conditions. Discussion of aeration delivery device options is provided in the following sections. Each basin will be equipped with an influent valve, a transfer pump, a mixer, a decanter, and a control package with adequate instrumentation to monitor the treatment process.

5.4.1.1 Option 1 – Mechanical Aeration

Mechanical aeration is the agitation of the wastewater surface to promote solution of air from the atmosphere. With the windy climate at the WVWRF site, water losses from wind-blown aerosol mist generated by the action of surface mechanical aerators throwing water into the air is considered a disadvantage. In general, mechanical aerators perform more efficiently in circular tanks than rectangular tanks. Mechanical aeration is not the preferred aeration option and will not be considered further.

5.4.1.2 Option 2 – Diffused Aeration

Diffused aeration uses a submerged aeration delivery device to introduce air into the wastewater and is generally split into two categories: coarse-bubble and fine-pore aeration. The large sized bubbles generated with coarse-bubble aeration are much less efficient at oxygen transfer and will not be considered further. Compressed air is passed through a porous media to produce the fine bubbles in fine-pore aeration.

Fine-pore diffusers are available in plates, panels, domes, discs, and tubes, and can be made of a range of materials including ceramics, porous plastics, and perforated membranes. Fine-pore aeration can transfer oxygen to the wastewater rapidly and is the most energy efficient alternative considered. With full-floor coverage, it will also provide all required mixing during the aeration phase of treatment.

Upon conversion to an MBR, the fine-pore aeration will continue to perform well. Additional blowers will be required and the airflow through the diffusers will increase to meet the additional aeration demands of the MBR system. The aeration will continue to provide all the requisite mixing in the aeration basin.

Over time, the small pores of the aeration delivery device begin to foul, gradually increasing the blower power demand. Periodic cleaning is required to refresh the fine-pore diffusers. To accomplish this, the basin will usually be drained and taken out of operation for a week or more. The frequency of cleaning varies from facility to facility, but is typically performed every 6-24 months.

5.4.1.3 Option 3 – Jet Aeration

Jet aeration combines compressed air with mixed liquor recirculation. A pumping system recirculates mixed liquor from the bioreactor, injecting it with compressed air within a specialized nozzle before releasing it into the bioreactor. The process shears the coarse bubbles into fine bubbles without the need for diffusers. During the aeration phase of treatment, jet aeration will provide all the necessary mixing. Additionally, this form of fine-bubble aeration has a low incidence of fouling and requires minimal cleaning and maintenance.

But jet aeration has a few shortcomings. It is less energy efficient than fine-pore diffusion. And the turbulence created by jet aeration delivery system can cause floc to shear that may inhibit sludge settling. Further, during the conversion to MBR, the aeration system would need to be converted to fine-pore aeration. Jet aeration is not full-floor coverage, it only provides point mixing. With the higher MLSS of MBR operation, there would be dead zones without oxygen in the aeration basin.

5.4.1.4 Comparison of SBR Aeration Alternatives

When considering SBR technology, the decision between fine-pore and jet aeration for the SBR is primarily a comparison of efficiency versus maintenance. Fine-pore aeration is more energy efficient, but the diffusers require shutting down the basin for routine cleaning. Jet aeration is less efficient, but requires little to no maintenance. A disadvantage of jet aeration is that it can cause floc to shear which could negatively impact settleability. Fine bubble aeration is compatible with potential conversion to

MBR. Fine-pore aeration is the preferred option. The advantages and disadvantages of each aeration option are shown in Table 5-3.

Table 5-3 – Comparison of SBR Aeration Alternatives

Technology	Advantages	Disadvantages
Fine-Pore Aeration	• Energy efficient • Provides adequate mixing even at high MLSS concentrations • Fine-bubble aeration • Will continue to perform well after MBR conversion	• Diffusers require routine cleaning to prevent performance degradation
Jet Aeration	• Minimal maintenance required • Provides adequate mixing at low to moderate MLSS concentrations • Fine-bubble aeration • Flexibility with two modes of operation: aeration with mixing, and mixing only	• Less energy efficient • Causes floc to shear which may inhibit sludge settling • Does not provide adequate mixing at high MLSS and will require upgrading to fine-pore aeration during MBR conversion

5.4.2 Aerobic Granular Sludge

Aerobic granular sludge (AGS) process is an emerging technology that is showing positive results by offering the potential of achieving better treatment, better settleability, and using less energy and less tank volumes than CAS. Currently marketed by Aqua-Aerobic Systems, Inc. under the trademark name AquaNereda, the process is a slight variation to the fill-and-draw process of a SBR whereby the influent flow is introduced slowly and uniformly across the bottom of the tank as the influent up-flows through the sludge blanket.

SBRs can be easily retrofitted to an AGS process. Conversion of SBRs to an aerobic granular sludge process could potentially increase capacity without adding more tankage. To accommodate a potential conversion to AGS in the future, the proposed SBR tank depth should have an operating depth of not less than 20 feet.

AGS is defined as an aggregate of microbial origin that holds together better and settles much faster than flocculent sludge without the need for biofilm carriers or media. Figure 5-6 compares settling characteristics of aerobic sludge granules (left) and CAS MLSS (right) after a five minute settling time.

With AGS, bioreactor biomass concentrations can operate higher than with SBRs which reduces reactor volumes and biosolids settle faster which reduces duration of SBR settling and decanting cycle times. Based on these factors, converting from SBR to AGS offers the potential to increase treatment capacity without adding new bioreactor tanks.

While more than 10 full-scale facilities are currently under design or construction in USA, GAS is a relatively new technology. On-going performance testing is expected to continue to generate information to more completely determine process stability, performance, and design criteria. The merits and benefits of GAS are will be more clearly established in the next 5-10 years. Implementing SBR at the WVWRF for the initial phase of the project will give MSWD another potentially viable option to consider when it comes time to upgrade and expand the treatment plant.

Figure 5-6 – Comparing Settling Characteristics with Aerobic Granular Sludge on left versus Conventional Activated Sludge on right (source: Aqua Aerobic Systems, Inc.)

5.4.3 Membrane Bioreactors

MBR systems combine activated sludge biological treatment with an integrated membrane filtration system to provide enhanced organics stabilization, nutrient removal, and suspended solids removal. The MBR system uses a low-pressure membrane filtration system (e.g., microfiltration or ultrafiltration) and eliminates the need for secondary clarifiers and tertiary filtration for liquid-solid separation. With the membrane units forming a "barrier" for separation of liquids and solids, MBR systems are designed to operate at MLSS concentrations as high as 8,000 to 10,000 mg/L, resulting in a much smaller aeration tank volume requirement compared to CAS. No additional treatment units are needed other than disinfection to produce tertiary disinfected recycled water. Elimination of secondary clarifiers and tertiary filters significantly reduces the overall footprint of the facility. Figure 5-7 presents a typical process schematic of an MBR system.

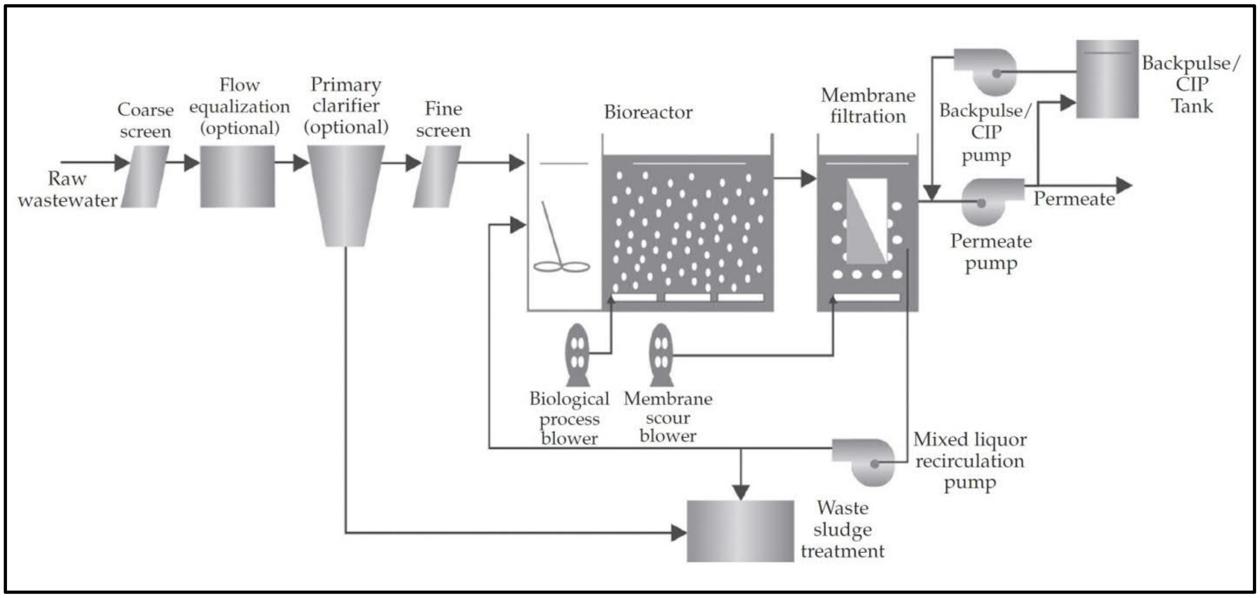


Figure 5-7 – Process Schematic of an MBR System (Adapted from WEF MOP No 36)

MBR systems require a fine screening with an opening of typically 1-2 millimeter in each direction to protect the membrane against abrasive particles and ensure membrane integrity. As in other activated sludge systems, primary clarification is optional. An MBR system also requires membrane tanks that house membrane modules; air scour system for membrane agitation and a clean in place (CIP) system for periodic membrane cleaning.

MBR systems may be classified based on membrane pore sizes (i.e. microfiltration vs. ultrafiltration), membrane materials (polymeric vs. composite or ceramic) and membrane element types (i.e. hollow fiber, flat sheet, tubular).

Although ultrafiltration membranes are usually more effective to remove small viruses, no correlation has been found between membrane pore size and organic, solids and nutrient removal performance.

MBR systems may also be categorized based on membrane element types that include hollow fiber, flat sheets and tubular membranes. Tubular membranes are tertiary membranes, not immersed, which can be installed downstream of a secondary process. One option of conversion of the SBR process would involve pumping flow from the SBR through tubular filters for polishing and virus removal. It would not increase plant capacity as the immersed membranes would, so it was not considered further for the future plant expansion.

Hollow fibers have an outside diameter varying between 0.5 and 3 mm. An element is formed by sealing the hollow fibers to one or two headers which may be circular or rectangular. The individual fibers are potted as several bundles. The elements are grouped together in a frame, rack or cassette which is operated as one unit.[1]

Flat sheet membranes are membrane envelopes where two membrane sheets are connected by internal support structure that serves as the permeate collection channel. The overall thickness of the membrane

[1] Water Environment Federation (WEF) Manual of Practice 36.

panels vary between 5 and 13 mm.[1] Figure 5-9 and Figure 5-8 shows hollow fiber membrane cassettes and flat sheet membrane modules, respectively.

Figure 5-9 – Hollow Fiber

Figure 5-8 – Flat Sheets

Table 5-4 compares membrane element types including their advantages and disadvantages.

Table 5-4 – Comparison of Membrane Element Types

Criteria	Hollow Fiber	Flat Sheet
Packing Density	High	Moderate
Operational Flux	Moderate	Moderate to high
Clogging Propensity	Moderate	Moderate to high
Turn Up/Down	Limited	Moderate
Cleaning Efficiency	Good	Poor
Track Record and Operational History	Excellent	Moderate
Energy Use	Moderate	Moderate to high
Capital Cost	Moderate	Moderate to high

Hollow fiber membrane element types are the most common configuration used in MBR applications with more than 15 commercial products available in USA market. Hollow fibers have the best track record and are more space efficient, energy efficient, and capital cost efficient than flat sheet MBR systems. Therefore, MBR sizing and costing in this report will be based on hollow fiber membrane elements.

5.4.4 Evaluation and Recommendation

Table 5-5 provides an evaluation of the advantages and disadvantages of SBR and MBR technologies.

Table 5-5 – Comparison of SBR and MBR Technology

Technology	Advantages	Disadvantages
SBR	• Lower capital cost • Lower operating cost • Simple operation • Fully automated • Compact Design • Reliable treatment to less than 10/10/10 BOD/TSS/TN • Can be upgraded to produce recycle water by either combining with disc filtration or converting to MBR • Emerging technology shows potential to increase capacity, improve performance, and improve energy efficiency by converting to granular activated sludge	• Add tertiary filtration and disinfection to produce recycle water • Larger basins are required for treatment compared to MBR
MBR	• Superior effluent quality (i.e., TSS<1.0 mg/L, turbidity<0.2 NTU) • Reliable and robust performance, much less impacted by influent water quality fluctuations • Sludge settleability and bulking problems are much less of a concern • Smaller activated sludge basin volume and overall footprint • No need for tertiary filtration to produce recycle water • Fully automated • Well-stabilized sludge; low biosolids production	• Higher capital cost • Higher energy requirements • Higher operating costs • Add fine screening (1-2 mm) upstream of MBR • Add disinfection to produce recycle water • If membranes are not properly maintained, fouling can affect MBR system capacity

Until water recycling is determined to be economically viable and in MSWD customers' best interests, there is no need to implement a wastewater treatment plant capable of producing recycle water. SBR technology with fine bubble diffusers was selected as the more appropriate and lower-cost alternative that meets the initial effluent discharge requirements and offers the most flexibility to accommodate various upgrades to produce recycle water in the future consistent with MSWD's long-term goal. The higher capital investment and higher operating costs for MBR are not justified as part of the initial phase of the proposed treatment plant.

5.5 Effluent Management

Treatment plant effluent will be disposed by discharging to infiltration basins. There are two options for infiltration basins: 1) onsite infiltration at the proposed plant site, and 2) offsite infiltration at an 80-acre site owned by MSWD located north of the proposed plant site. Figure 5-10 shows the locations of the proposed treatment plant site, a potential offsite infiltration basin site, and locations of MSWD public water supply wells.

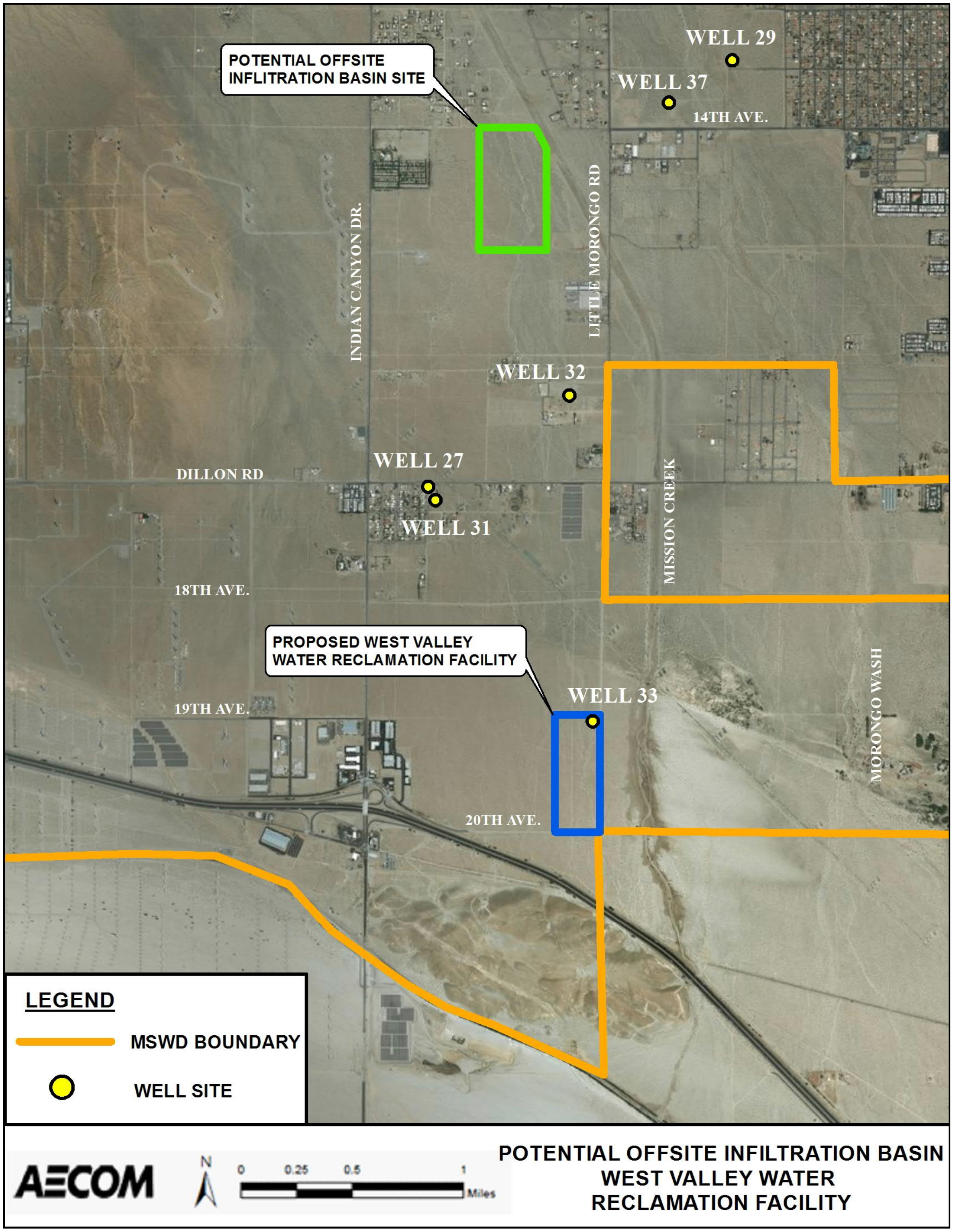


Figure 5-10 – Location Map for Proposed WVWRF and Potential Offsite Infiltration Basin Site

5.5.1 Onsite Infiltration Basin

Infiltration basins are proposed to be located onsite in the southern portion of the proposed site. The proposed wastewater treatment plant is situated over the Garnet Hill groundwater sub basin. Infiltration basins are currently being used at Horton WWTP.

Site master planning for the proposed treatment plant will allow for a future effluent pump station to transfer effluent offsite. An effluent pump station is not currently planned as part of the initial plant improvements.

5.5.2 Offsite Infiltration Basin

MSWD owns an 80-acre site located north of the proposed treatment plant that could potentially be used to locate infiltration basins. This 80-acre site is situated over the Mission Creek groundwater sub basin.

AECOM performed preliminary geotechnical testing of the 80-acre site. Findings show that the soils at the second site have percolation rates similar to the proposed treatment plant site. The basis of design for sizing infiltration basins at the 80-acre site is expected to be the same as the basis of design for the proposed treatment plant site. The only difference is that a new effluent pumping system would be required to deliver effluent from the proposed treatment plant to the 80-acre site.

5.5.3 Evaluation and Recommendation

During the development of this report, groundwater modeling was performed that demonstrates the proposed treatment plant site is suitable for locating infiltration basins.

Considering that offsite infiltration basins would require additional cost for installation and operation of an effluent pump station and conveyance pipeline, and will require similar-sized basins, the proposed infiltration basins are recommended to be located at the treatment plant site.

5.6 Solids Thickening Options

Thickening is a physical separation process used to increase the solids content of sludge by removing a portion of the water. Thickening processes generally increase the total solids (TS) concentration from 1% or less up to 4 to 8% TS concentration. This would increase the capacity of downstream digestion and dewatering processes. Thickening is generally accomplished by either gravity sedimentation and decanting or by mechanical means such as gravity belt thickening or rotary drum thickening. Most all mechanical thickening technologies presented in this report will typically capture 95 to 98% of the solids which benefits the treatment plant by minimizing loadings from side streams returned to the wastewater treatment process.

5.6.1 Option 1 – Aerated Sludge Holding Tank with Decanter

Waste activated sludge (WAS) can be thickened by gravity in the aerated sludge holding tank, through the use of a decanter that removes supernatant. During normal operation, the aerated sludge holding tank would be aerated using coarse bubble diffusers. Process air would be sized for mixing purposes to keep solids suspended in the tank. Under the thickening operation, the aeration system would be shut off and solids allowed to settle. This would occur periodically when required to achieve the desired thickened solids concentration. Similar to an SBR process, in the thickening phase, air would be shut off to allow solids to settle. Following the settling phase, supernatant would be decanted and returned to the front end of the plant. This would thicken the WAS in the storage tank without additional thickening equipment required. The plant team could undertake this process as often as is required to achieve the desired solids concentration in the tank.

There would be two aerated sludge holding tanks for redundancy if one is taken offline. The decanter would be a similar installation to the SBR decanter. Aeration/ settling times and supernatant pumping rate would be determined based on WAS flow rates, sludge settling rates and desired concentration in the storage tank.

Gravity thickening would be relatively labor intensive as it would require the plant to monitor the process during decanting. A dedicated mechanical thickening process would be less labor intensive. Thickening in an aerated sludge storage tank is generally suitable for smaller treatment plants with capacities typically of less than 1.5 mgd. In the future when the plant flows begin to approach 1.5 mgd, a dedicated thickening process could be warranted.

The major benefit of gravity thickening is that it allows for thickening during initial lower flow conditions without dedicated thickening equipment. This would reduce capital and operating costs during the first few years of plant operation.

5.6.2 Option 2 – Gravity Belt Thickener

Gravity belt thickeners (GBTs) are commonly used for thickening WAS. Figure 5-11 shows a photograph of a GBT. The GBT uses a porous polyester belt that travels along a series of rollers. The sludge is first conditioned with cationic polymer before being fed into a feed/distribution box where the sludge is distributed evenly across the width of the moving belt on the top of the unit. As the belt moves forward, the sludge passes through a series of polyester plows that open channels for draining free water from the sludge solids. The concentrated solids are dropped off the discharge end of the unit into a hopper or chute. Belt travel speeds are slow to minimize wear. GBTs are available in open frame or enclosed designs. The enclosed design reduces odor potential, but is more difficult to clean, repair and maintain. GBTs are available in sizes ranging from 0.5-meter effective belt width to 3.0-meter effective belt width in increments of 0.5 meter. The GBT typically produces thickened sludge that averages 4 to 8% TS concentration. When thickening raw WAS, the polymer dosage typically varies from 4 to 6 pounds (lb.) per dry ton (dt).

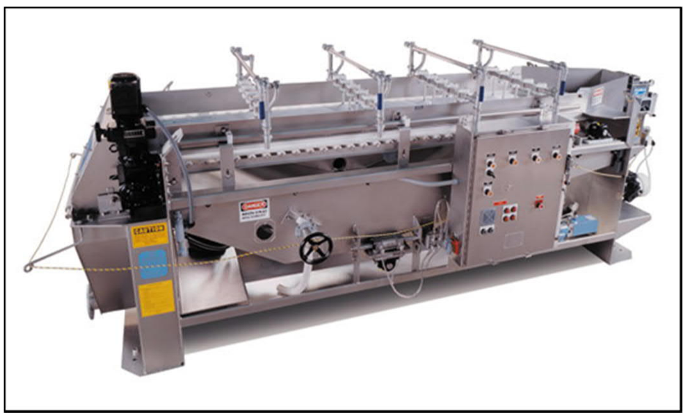


Figure 5-11 – Enclosed Gravity Belt Thickener Illustration

The total power required to drive the belt is 1.0 horsepower (HP). The belt tracking assembly that operates intermittently is 1.0 HP. The wash water pump, which operates continuously, has the capacity of approximately 20 gpm at 105-foot total dynamic head and is driven by a 2.0 HP motor. The total connected load is 4.0 HP.

There are numerous vendors who offer GBT technology, including Ashbrook/Alfa Laval, BDP, Andritz, Komline Sanderson, and others.

5.6.3 Option 3 – Rotary Drum Thickener

Rotary drum thickeners (RDTs) are available for thickening WAS. Figure 5-12 shows an illustration of a typical RDT unit. The sludge is conditioned with a cationic polymer and is fed into the center of the rotating mesh screen drum. The sludge passes through the unit from one end to the other and the water drains out through the bottom of the mesh screen. The concentrated solids are dropped out of the discharge end of the unit into a hopper or chute. RDT rotating speeds are slow to minimize wear. The enclosed design of the technology reduces odor potential. RDTs are available in capacity ranging from 25 to 400 gpm per unit. The RDT typically produces thickened WAS that averages 4 to 7% TS concentration. When thickening raw WAS, the polymer dosages typically vary from 10 to 15 lb./dt. The power required to drive the drum is 1.5 to 2 HP and the wash water pump would be approximately 2.0 to 3.0 HP depending on the size of the unit selected. Wash water is used intermittently with an RDT to spray off the drum.

Figure 5-12 – Rotary Drum Thickener Illustration

There are numerous vendors who offer the RDT technology, including Parkson, Ashbrook, FKC, JDV, JWC, and others.

5.6.4 Option 4 – Disc Thickener

The disc thickener uses a rotating wiper assembly to concentrate solids. Figure 5-13 shows an illustration of a typical disc thickener unit. The sludge is conditioned with a cationic polymer and pumped into a stationary stainless steel chamber that has an inclined stainless steel mesh screen floor that allows filtrate to drain out of the unit. The wiper assembly moves the concentrated solids up the incline to a chute where they are dropped into a pump, conveyor, or hopper/holding tank. Rotational speeds are slow to minimize wear. Filtrate from the unit can be used as wash water if a booster pump is provided. The enclosed design of the technology reduces odor potential. The disc thickener typically produces thickened sludge that averages 4 to 6% TS concentration. When thickening raw WAS, the polymer dosage typically varies from 10 to 15 lb/dt. The wiper assembly is driven by a 0.75 HP motor. The wash water pump, which operates intermittently, is driven by a 2.0 to 3.0 HP motor. The total connected load is 2.75 to 3.75 HP depending on the capacity of the selected unit. This technology is offered by Huber Technologies and others.

Figure 5-13 – Disc Thickener Illustration

5.6.5 Evaluation and Recommendation

A comparison of mechanical thickening technologies is provided in Table 5-6.

Table 5-6 – Comparison of Mechanical Thickening Technologies

Technology	Advantages	Disadvantages
Gravity Belt Thickener	• Lower polymer use (4-6 lb/dt) • More suited to SBR • Combined GBT/BFP possible	• More complex, more moving parts • Higher installed cost • Larger footprint
Rotary Drum Thickener	• Lower installed cost • More suited to MBR • Smaller footprint • Enclosed unit (less odor) • Simple, fewer moving parts	• Higher polymer use (10-15 lb/dt)
Disc Thickener	• Small footprint • Enclosed unit (less odor) • Low operating costs • Slow rotation, low energy use	• More complex, more moving parts • Higher installed costs

The preferred technology for thickening is RDTs. Rotary drum thickening technology is simple with less moving parts and lower installed cost, low operating cost, and is a reliable technology for thickening WAS. RDTs would be configured to thicken raw WAS from the secondary process or from the aerated sludge storage tank. Dedicated thickening will reduce the volume of waste sludge that must be either pumped or transported to a dewatering process.

During the first few years of operation, when plant flows are expected to be relatively low, it is recommended that waste activated sludge be thickened by gravity using the aerated sludge storage tanks and decanters. This is a lower cost alternative and is intended as an initial solution until such time that the total plant flows increase to a rate that can justify mechanical thickening.

5.7 Solids Dewatering Options

Dewatering is a physical separation process aimed at reducing the moisture content of sludge and biosolids. Dewatering involves a higher level of moisture removal than thickening. TS concentrations ranging from 15 to 30% can typically be achieved in municipal solids dewatering applications, depending on sludge characteristics, dewatering device, and polymer dosage. Dewatering can be the final stage of sludge and/or biosolids processing before hauling and disposition, or it can be an intermediate process that precedes another process such as drying or incineration. Dewatering can be accomplished by mechanical means using equipment such as centrifuges, BFPs, rotary presses, or screw presses. All of the dewatering technologies listed typically capture 95 to 98% of the solids to minimize impacts of the filtrate on the existing wastewater treatment processes.

5.7.1 Option 1 – Belt Filter Press

BFPs are commonly used for dewatering raw or digested solids. Figure 5-14 shows a photograph of a BFP. The BFP uses a porous polyester belt that travels along a series of rollers. The sludge is first conditioned with cationic polymer before being fed into a feed/distribution box where the sludge is distributed evenly across the width of the moving belt on the top of the unit (gravity zone). As the belt moves forward, the sludge passes through a series of polyester plows that open channels for draining free water from the sludge solids. The concentrated solids are dropped off the end of the gravity zone into the pressure zone where pressure is applied to dewater the solids. The pressure is mechanically applied to solids sandwiched between two tensioned belts passing through a series of varying diameter rollers that sandwich (or trap) the solids between the two belts. The pressure forces liquid to drain out through the bottom of the porous belt where the liquid (filtrate) is collected in a series of pans and drained away. For a given belt tension, as the roller dimension decreases, increasing pressure is exerted to the sludge, removing more water. The units can be furnished with either an open frame or an integral enclosure that is part of the BFP frame. The integral enclosure is intended to contain odors and aerosols from the belt wash and filtrate drainage areas to provide a safer work environment for the operators in the process area.

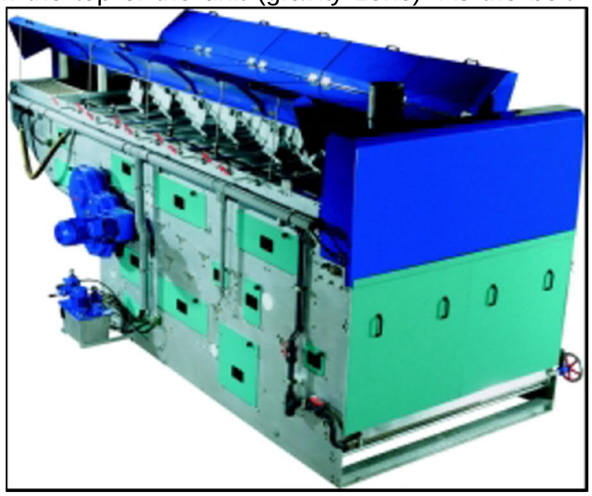


Figure 5-14 – Enclosed Belt Filter Press Illustration

BFPs are available in widths ranging from 0.5 to 3.0 meters (in 0.5 meter increments). BFP units are available in “standard” and “high-solids” configurations. The “standard” configuration BFP has 7 to 9 rollers within the pressure zone. The “high solids” configuration BFP has 12 to 14 rollers within the pressure zone. Standard BFP units typically produce dewatered cake with a TS concentration of approximately 15 to 18% when dewatering raw or digested WAS. High solids BFP units typically produce dewatered cake with a TS concentration of approximately 16 to 20% when dewatering raw or digested WAS. The polymer dosage rate typically varies between 15 to 25 lb/dt. Performance results depend on sludge feed concentration. The power requirements include the following:

- Belt drive: 2 HP (operates continuously)
- Belt tracking unit. Either hydraulic or pneumatic systems are used: 0.5 HP (operates intermittently)
- Wash water booster pump: 2 HP (operates continuously)

Power and wash water requirements would be higher for the wider belts.

There are numerous vendors who offer BFP technology, including BDP Industries, Komline Sanderson, and many others.

5.7.2 Option 2 – Combination Gravity Belt Thickener/ Belt Filter Press

Combination GBT and BFP (“three-belt BFP”) units are commonly used for dewatering raw or digested WAS. Figure 5-15 shows a photograph of a three-belt BFP. The three-belt BFP uses porous polyester belts that travel along a series of rollers. The sludge is first conditioned with cationic polymer before being fed into a feed/distribution box where the sludge is distributed evenly across the width of the moving belt on the top of the unit (gravity belt thickening zone). A separate belt and drive is provided for the gravity zone. As the gravity zone belt moves forward, the sludge passes through a series of polyester plows that open channels for draining free water from the sludge solids. The concentrated solids are dropped off the end of the gravity zone into a separate pressure zone with a second drive motor where pressure is applied to dewater the solids. The pressure is mechanically applied to solids sandwiched between two tensioned belts passing through a series of varying diameter rollers that sandwich (or trap) the solids between the two belts. The pressure forces liquid to drain out through the bottom of the porous belt where the liquid (filtrate) is collected in a series of pans and drained away. For a given belt tension, as the roller dimension decreases, increasing pressure is exerted to the sludge, removing more water. The units can be furnished with either an open frame or an integral enclosure that is part of the BFP frame. The integral enclosure is intended to contain odors and aerosols from the belt wash and filtrate drainage areas to provide a safer work environment for the operators working in the process area.

BFPs are available in widths ranging from 0.5 to 3.0 meters (in 0.5 meter increments). Three-belt BFP units typically produce dewatered cake with a TS concentration of approximately 16 to 19% when dewatering raw or digested WAS. The polymer dosage rate typically varies between 15 to 25 lb/dt. Performance results depend on sludge feed concentration. The power requirements include the following:

- Gravity zone belt drive: 2 HP (operates continuously)
- Pressure zone belt drive: 2 HP (operates continuously)
- Belt tracking unit. Either hydraulic or pneumatic systems are used: 1.0 HP (operates intermittently)
- Wash water booster pump: 15 HP (operates continuously)

Power and wash water requirements would be higher for the wider belts.

This technology is offered by BDP Industries and others. This technology is currently in use at the Horton WWTP.

Figure 5-15 – 3-Belt BFP Illustration

5.7.3 Option 2 – Centrifuge

A centrifuge uses a rotating steel housing to concentrate solids. Figure 5-16 shows a photograph of a centrifuge. The sludge is conditioned with a cationic polymer and pumped into the rotating steel cylinder (bowl). Rotational speeds are typically several thousand rpm. The centrifugal forces created by the rotation push the dense solids toward the outer wall of the cylinder. A separate screw conveyer, known as a scroll, inside the rotating cylinder conveys concentrated solids to the outlet. Centrate is removed at the opposite end of the unit. Centrifuges are available in various sizes with capacities ranging from approximately 25 to 1,000 gpm. The centrifuge unit typically produces dewatered cake with a TS concentration ranging from 17 to 21% TS when dewatering raw or digested WAS. A centrifuge does not require wash water, and because they are compact and enclosed, odor control is reduced compared to other technologies where sludge dewatering is open to the atmosphere. The polymer dosages typically vary from 30 to 50 lb/dt when dewatering raw or digested WAS, depending on the sludge feed concentration. Due to the high speed, centrifuges use more power and have increased O&M requirements. A 40 HP motor would be required to rotate the bowl in each unit. In addition, a 10 HP backdrive motor is needed to operate the scroll. The total connected HP would be 50 HP. Operator attendance is expected to increase initially when the operators are gaining familiarity with the new system. The high-speed operation of the centrifuge units may increase cost and labor for repair and replacement of parts.

Figure 5-16 – Centrifuge Illustration

In general, the performance of centrifuges is better than Belt Filter Presses and Rotary Presses when dewatering MBR sludge.

There are numerous vendors who offer centrifuge dewatering technology, including Alfa Laval, GEA Westfalia, and others.

5.7.4 Option 3 – Rotary Presses

A rotary press is an enclosed unit with a painted steel housing. Figure 5-17 shows a photograph of a rotary press. A rotary press operates with a rotating stainless steel channel. Rotation is typically slow, between 0.5 and 1.5 rpm. Sludge conditioned with polymer is fed into a rectangular channel that rotates between two parallel revolving stainless steel screens. The flocculated sludge dewaters as it advances within the channel and filtrate passes through the screens. Eventually, the sludge forms a cake near the outlet side of the press. The frictional force of the slow-moving screens and the controlled outlet restriction help dewater the cake before extrusion. The rotary press does not require a continuous supply of wash water. Rotary presses are available with one, two, four, or six modules. Feed rates for a single module typically range from 10 to 15 gpm depending on the feed concentration. The throughput performance of a six-channel rotary press unit is approximately 60 to 90 gpm, depending on the feed concentration. A 15 HP drive motor is required to operate a four channel unit. A 3 HP flocculation mixer is also required. The total connected HP would be

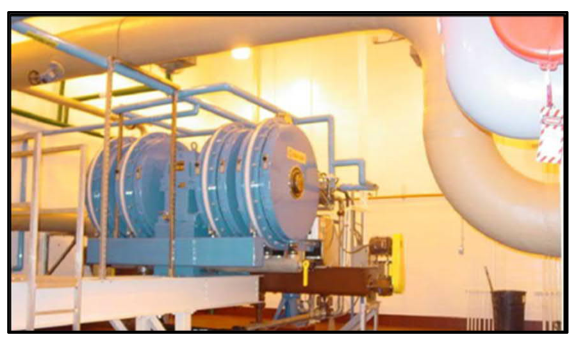


Figure 5-17 – Rotary Press Illustration

18 HP. A rotary press typically achieves cake with a TS concentration of 13 to 17% when dewatering raw or digested WAS. The polymer dosage rate typically varies between 35 to 45 lb/dt.

5.7.5 Option 4 – Screw Presses

A screw press is housed in a stainless steel enclosure and uses a conical stainless steel screw shaft and cylindrical stainless steel sieve. Figure 5-18 shows a photograph of a screw press. Sludge is dewatered in three zones within the press. In the inlet/drive zone, filtrate is removed using the pressure produced by the feed pump via a large free filter surface area. A sensor in this zone regulates the pressure. In the thickening/dewatering zone, the volume of the sludge between the flights of the screw is reduced by the conical screw and the sludge is pressed against the inner screen surface, forcing out water. Dewatering occurs in the press zone by a pneumatic counter-pressure cone at the press discharge. Residence time in the press is controlled by the speed of the screw. Rotational speeds tend to be low, less than 10 rpm. Screw presses are available with diameters ranging from 0.28 to 1.25 meters (11 to 49 inches). The throughput capacity of screw presses typically range from approximately 10 to 125 gpm depending on the diameter selected and the feed concentration. The screw press would be driven by a 3 HP electric motor. The screw press typically produces between 15 and 19% cake when dewatering raw or digested WAS, depending upon sludge feed concentration. The power and wash water requirements are similar in magnitude to that of a BFP; however, the wash cycle is intermittent for a screw press. The odor control is minimized because the units are enclosed. The polymer dosage rate typically varies between 25 to 40 lb/dt when dewatering raw WAS.

Figure 5-18 – Screw Press Illustration

This technology is offered by Huber, FKC, Schwing-Bioset, Ishigaki, and others.

5.7.6 Evaluation and Recommendation

A comparison of dewatering technologies is provided in Table 5-7.

Table 5-7 – Comparison of Dewatering Technologies

Technology	Advantages	Disadvantages
Belt Filter Press	• Low energy use • Lower installed cost	• High polymer use • Larger footprint • Lower dewatered solids concentration • Open (more odorous) • Requires wash water
Combined GBT & BFP	• Used at Horton WWTP • Low energy use • Higher dewatered solids concentration than BFP	• High polymer use • Larger footprint than BFP • Lower dewatered solids concentration • Open (more odorous) • Requires wash water

Technology	Advantages	Disadvantages
Rotary Press	• Low energy use • Small footprint • Enclosed (less odorous)	• High polymer use • Lower dewatered solids concentration
Screw Press	• Low energy use • Small footprint • Higher dewatered solids concentration • Enclosed (less odorous)	• High polymer use • Requires wash water • Less effective on activated sludge
Centrifuge	• Lower polymer use • Small footprint • Highest dewatered solids concentration • No wash water required	• Higher energy use • More odorous sludge • High maintenance cost

The preferred technology for dewatering is a combined GBT and BFP (3-Belt BFP). The combined technology allows for higher dewatered cake solids concentration than the BFP alone. This technology is in use at the Horton WWTP. MSWD has indicated their preference to match the technology used at the Horton WWTP.

5.8 Solids Stabilization and Disposal

Options for solids stabilization and disposal are evaluated for the new WVWRF. For processes without primary clarification, anaerobic processes are not efficient, and add little value to the treatment plant. Two options below are viable for the type of treatment. The following alternatives are considered:

- Option 1 – Aerated Sludge Storage
- Option 2 – Conventional Aerobic Digestion

5.8.1 Aerated Sludge Storage

The existing Horton WWTP currently dewaters WAS using a “3-belt BFP”. The mechanically dewatered solids are loaded into rented trailers. Once a trailer is full, the dewatered solids are transported to a contract disposal facility. Current disposal cost is approximately $53 per wet ton.

This alternative was considered for the new WVWRF. WAS would be stored in aerated sludge storage tanks. Process air would be provided to the storage tank for mixing and to prevent the solids from turning septic. The solids retention time (SRT) of the sludge storage tanks will not be long enough for any significant solids stabilization to occur. Aerated WAS from the sludge storage tank would be dewatered using the preferred dewatering technology, and dewatered solids trucked offsite to the same contract disposal facility as per Horton WWTP.

5.8.2 Aerobic Digestion

Aerobic digestion involves the biological decomposition of organic matter and assimilation of inorganic matter into biomass in the presence of oxygen. A general schematic of an aerobic digester is provided in Figure 5-19 The process is similar to conventional extended aeration activated sludge biological treatment. The process works when the supply of available food or substrate is depleted and the microorganisms begin to consume cell tissue to obtain energy (endogenous phase). In the process, the degradable solids are oxidized to carbon dioxide and ammonia. As digestion proceeds, nitrification occurs and the ammonia is oxidized to nitrate. The nitrification process consumes alkalinity. A portion of the alkalinity can be recovered if anoxic cycling is incorporated into the operation of the system during decanting, allowing denitrification. A portion of the nitrate is converted to nitrogen gas. The aerobic digestion process can be a continuous or batch process, and conventional aerobic digestion typically generates EPA Class B biosolids (if both the air supply and retention time are sufficient).

Aerobic digestion occurs when sludge is agitated with air or oxygen to maintain aerobic conditions for a mean cell residence time and temperature between 40 days at 20 degrees Celsius (°C) and 60 days at 15°C. Feed solids concentrations typically range from 2 to 6% TS. Volatile solids (VS) destruction typically varies from 38 to 45% if all aeration and mean cell residence time criteria are met. Supernatant, filtrate, or centrate from the dewatering process downstream of an aerobic digester can contain high concentrations of organic matter and nutrients that can create a detectable side stream load to the WWTP influent. Separate pretreatment for the removal of nitrogen and phosphorus is sometimes necessary at WWTPs that have stringent effluent nutrient limits in the waste discharge permit.

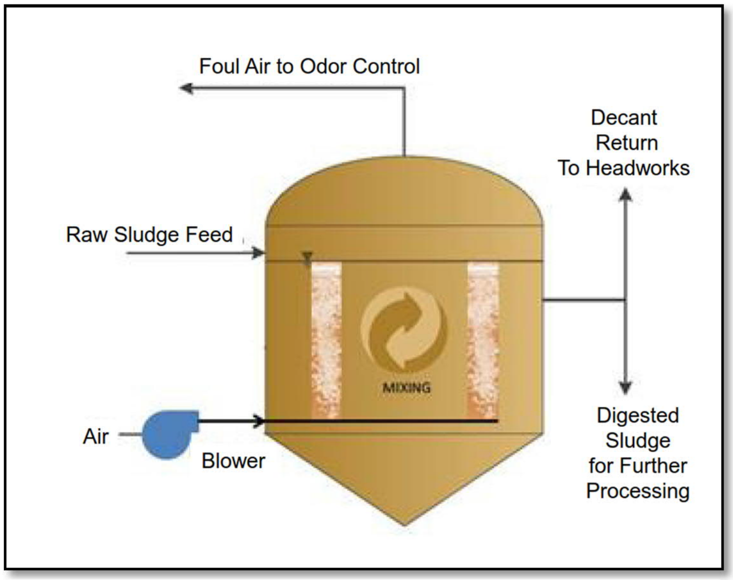


Figure 5-19 – Conventional Aerobic Digester

Advantages of aerobic digestion include:

- Volume reduction through VS destruction
- Digested dewatered cake can be distributed as Class B or disposed in landfill

Disadvantages of aerobic digestion include:

- High capital cost for new digesters;
- High energy cost;
- Relatively complex process to operate at a small facility;
- Can produce odors that are difficult to treat;
- Foam control is necessary; and
- End product is unappealing without further processing (such as composting or thermal drying).

5.8.3 Evaluation and Recommendation

The following section summarizes the advantages and disadvantages of the options for solids stabilization and disposal at the new WVWRF.

Aerated sludge storage is the least cost and simplest method for solids disposal. This option requires construction of an aerated sludge storage tank with a coarse bubble diffused aeration system (for mixing and suspension of solids). This will require compressors housed in a building for noise attenuation. Raw sludge would be dewatered using the preferred dewatering method and trucked offsite for disposal. This method of solids disposal is currently undertaken at the Horton WWTP and could be extended to the new WVWRF.

Aerobic digestion would require the construction of an aerobic digester with a diffused aeration system and process building to house the blowers. Aerobic digestion would produce Class B Biosolids for distribution or disposal in landfill. This process has a high capital and operating cost. However, the tipping fee for disposal of aerobically digested dewatered solids is anticipated to be lower than raw sludge.

Given that the proposed treatment plant is relatively small, the preferred method of solids stabilization and disposal is aerated sludge storage and dewatering as is currently practiced at the Horton WWTP. This is the least cost option. This will require a vendor to continue accepting unstabilized biosolids, which has not been a problem at the Horton WWTP. Other processes may be reconsidered in the future as flows begin to increase.

During a workshop, MSWD stated their preference for a Serpentix conveyor, dewatered sludge storage bin (silo) with live bottom hopper and platform scale. For the purpose of this report, we assumed a two-level building with direct loading to a truck trailer unit as the basis of space planning. The space planning is adequate to support the installation of the preferred units with some changes to structural components. Design details for the preferred units will be developed during final design.

5.9 Odor Control

Odor control is a process aimed at reducing the concentrations of hydrogen sulfide and other airborne contaminants in the air exhausted from the liquid treatment and solids handling processes proposed for the MSWD WVWRF. The principal sources of odor in the plant are expected to be:

- Influent pump station wetwell;
- Mechanical screening and grit removal areas; and
- Solids processing and storage.

Where covers are used, the trapped gases would be collected and treated. Odor control systems typically remove 95 to 99% of airborne hydrogen sulfide (H_2S). The following odor control systems were considered for the MSWD WVWRF:

- Chemical Scrubber;
- Biotrickling Filter;
- Biofilter;
- Activated Carbon; and
- Two Stage Systems.

5.9.1 Option 1 – Single Stage Chemical Scrubber

Chemical scrubbers achieve high odor-removal efficiency for a wide range of odorant loadings. See Figure 5-20 for an illustration of a chemical scrubber.

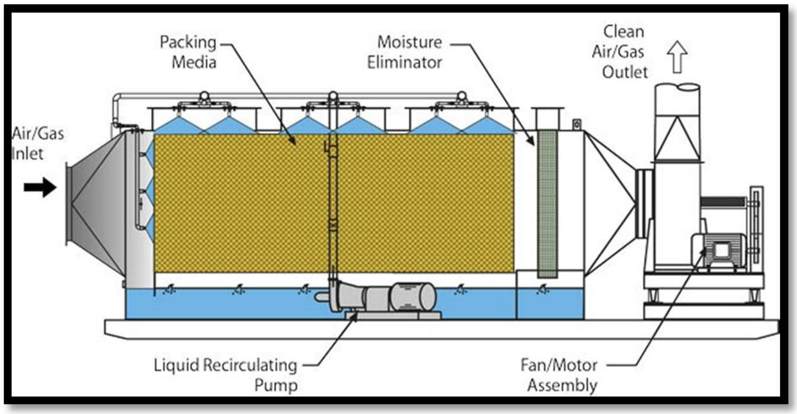


Figure 5-20 – Single Stage Chemical Scrubber

Proper pH and oxidation-reduction potential (ORP) controls are necessary to keep pH in the 9.5 to 10.5 range and ORP in the 400 to 800 range. At lower pH and ORP, the removal efficiency decreases. An additional low pH stage (2.0 to 3.0 pH) is required for removal of ammonia or amine based compounds. Wet chemical scrubbers consist of a corrosion-resistant vessel where odorous air contacts a liquid chemical solution that is sprayed from nozzles. The liquid solution is continuously recirculated through the packing material. Makeup water is added to flush reacted materials out of the system to optimize treatment. If makeup water is insufficient, the reacted chemicals reach a point of saturation and removal efficiencies are reduced, increasing outlet odor concentrations.

The advantages of a single stage chemical scrubber include:

- Achieves 99% removal efficiency of H_2S
- Available in a wide range of sizes with single systems from 50 cfm units to 40,000 cfm units
- Can treat air with velocity up to 400 feet per minute (ft/min)
- Footprint of a chemical scrubber is small compared to biological treatment
- Can be turned on and off to treat odors only when they are detected

The disadvantages of a single stage chemical scrubber include:

- Less effective on organic sulfur compounds
- Requires on-site chemical storage
- Requires the most sophisticated control and instrumentation
- Chemicals in waste blowdown can increase effluent TDS

5.9.2 Option 2 – Biotrickling Filter

Bio-trickling filters use media that allows the odorous compounds in the air stream to contact with an active biofilm and degrade. See Figure 5-21 for an illustration of a bio-trickling filter.

Water from the sump continuously recirculates, trickling through the inorganic or synthetic plastic media to keep the biofilm moist and biologically active. The water is kept at a pH of approximately 2.0 by adding only a small amount of makeup water. An acidic bacteria culture develops at the normal operating pH of 2.0 that effectively removes H_2S and some reduced sulfur compounds.

Figure 5-21 – Biotrickling Filter Illustration

The advantages of a biotrickling filter include:

- Low operational cost
- On-site chemical storage is not required
- Effectively treats H_2S concentrations in a range of 10 to 400 ppm. Ninety percent of ammonia and amines can be removed as a secondary effect.
- Available in a wide range of sizes with single systems capable of treating 50 cfm to 30,000 cfm.

The disadvantages of a biotrickling filter include:

- High capital cost
- Are not recommended if a plant has low, or highly variable H_2S concentrations.
- Not suitable for intermittent operation; must be operated continuously.

5.9.3 Option 3 – Biofilter

A biofilter consists of an open bed of inorganic media for the odorous air to pass through. See Figure 5-22 for an illustration of a biofilter.

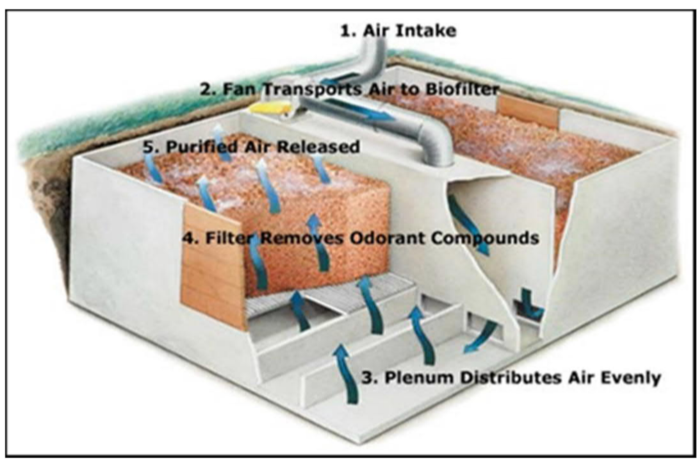


Figure 5-22 – Biofilter Illustration

As odors move through the media, sorption and bioconversion occur. Microorganisms oxidize the gases and renew the treatment capacity of the media. The inlet air flows through a humidification chamber to provide a relative humidity of nearly 100% before air enters the biofilter media bed. After humidification, the foul air is introduced into the bottom of the media. The bottom zone of the inorganic media provides an environment that is suitable for an acidic bacteria culture to develop at a normal operating pH of 2.0, effectively removing H_2S, and some reduced sulfur compounds. As the air moves up through the bed, a neutral pH (6.5 to 7.5) zone forms within the media. This allows a more diverse bacteria culture to grow, providing removal of a wider range of contaminants, including any remaining compounds not removed in the lower zone, such as ammonia, mercaptans, amines, ketones, volatile fatty acids, and some volatile organic compounds (VOCs).

The surface of the bed is irrigated intermittently to keep the media bed moist. Water that percolates through the media bed is collected and conveyed to the plant sewer. Air distribution through the organic media and proper moisture content is essential for microorganism activity.

The advantages of a biofilter filter include

- Low Capital cost
- Effective removal of H_2S
- Can handle the removal challenge of diverse mix of complex odors and VOCs better than any other odor control technologies

The disadvantages of a biofilter filter include

- Requires a larger area compared to chemical or biological scrubbers
- Subject to breakthrough and reduced efficiency
- Requires access for heavy equipment to perform media change-out
- Has to be operated continuously

5.9.4 Option 4 – Activated Carbon

Activated carbon reduces odor through a process that consists of a corrosion-resistant vessel containing activated carbon media that attracts and retains gases and particles on its porous surface. See Figure 5-23 for an illustration of an activated carbon scrubber.

Activated carbon can be produced from a variety of materials, such as wood, coal, coconut, nutshells, and petroleum-based products. The raw material is heated or treated to increase the material's adsorptive potential. Spent carbon is removed by vactor truck when saturated (near breakthrough) and replaced.

Figure 5-23 – Activated Carbon Illustration

5.9.5 Option 5 – Two Stage System

Two stage systems involve a combination of odor treatment technologies, commonly consisting of a biofilter followed by activated carbon for polishing. This allows for capture and reduction of organic and inorganic odors by combining gas adsorption and biological oxidation. After odors are treated with a biofilter, airflow is directed to a second stage that removes any VOCs or organic compounds that were not removed during the first stage. Two stage systems are able to combine the ability of biofilters and activated carbon to ensure complete removal of odors.

5.9.6 Evaluation and Recommendations

It is recommended to select an odor control system based on the expected airflow of the areas requiring odor control. The estimated airflow that must be treated at the WVWRF is 2,300 cfm based on 6 air changes per hour in odor producing facilities. As a general rule-of-thumb, activated carbon systems are typically used for systems with less than 10,000 cfm of air flow. Activated carbon systems are simple to operate and maintain. As plant flows continue to increase and more treatment processes are added, facilities, other systems such as a biofilter, may become more justified.

For the proposed treatment plant, the preferred method of controlling odors will be a system consisting of covers over selected process areas, foul air collection, and activated carbon treatment before exhausting to atmosphere.

Section 6 – Description of Recommended Facilities

6.1 General

The WVWRF construction would be designed to remove conventional pollutants (BOD_5 and TSS) and TN from the wastewater for disposal to onsite infiltration basins. Provisions will be made in the facility's initial design to accommodate future upgrades to tertiary treatment and to produce recycle water.

The following sections describe the recommended facilities, as listed below.

- Influent pump station
- Coarse screening with screenings compactor
- Vortex grit removal with grit classifiers
- Sequencing batch reactors (SBR)
- Effluent disposal to infiltration basins
- Aerated sludge storage with decanter for gravity thickening
- Rotary Drum Thickener
- 3-Belt Belt Filter Press (GBT + BFP) for biosolids dewatering
- Contract disposal of biosolids
- Odor control

Each facility includes a description for Phase 1 design and Future design. Phase 1 design is for average maximum monthly flow of 1.5 mgd. Future design is for average maximum monthly flow of 3.0 mgd.

The process flow diagram (PFD) for the recommended facilities is provided in Figure 6-1.

Of the recommended facilities, some could be deferred until later as plant flows increase over time, including:

- Rotary Drum Thickener
- Belt Filter Press (GBT + BFP) for biosolids dewatering

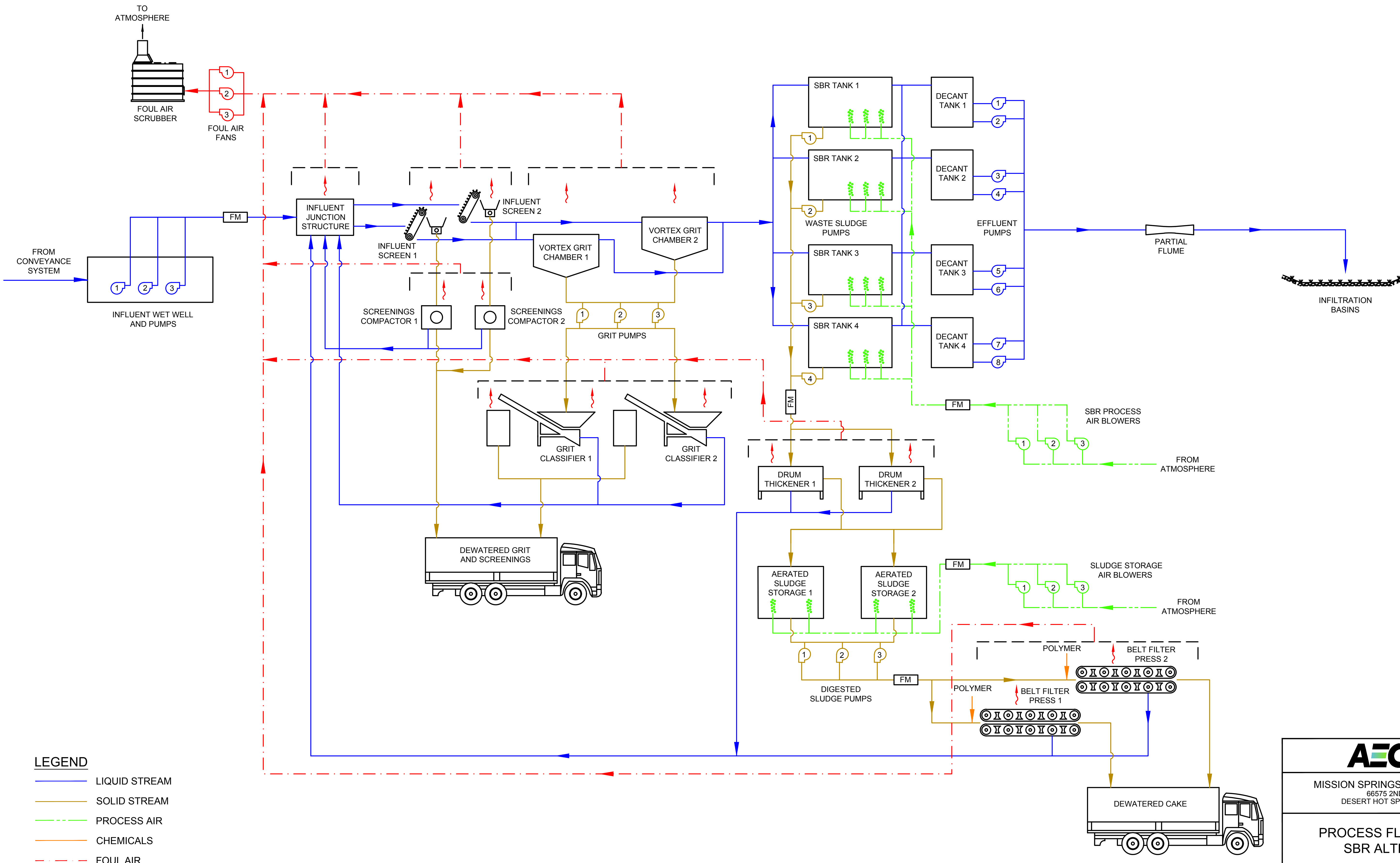
See Final Value Engineering Technical Memorandum for details on the potential cost reductions of deferring equipment.

6.2 Influent Pump Station

For influent pumping, a submersible pumping system was selected as the preferred alternative. This section discusses the design considerations for the influent pump station.

6.2.1 Phase 1 Design

The Phase 1 influent pump station (IPS) would be designed to pump a peak flow of 2,600 gpm (3.75 mgd). Three new pumps would be installed for the initial Phase 1 flows. Two pumps would be duty pumps, and one pump would be on dedicated standby. Preliminary pump selection is 6-inch submersible pumps with 30 HP motors, and VFDs. The pumps would be rated 1,300 gpm at 1,800 revolutions per minute (rpm), 10-inch impeller diameter, and 68% pump efficiency at the duty point. A magnetic flow meter would be provided to indicate and transmit influent wastewater flow data. An automatic sampler would also be installed at the IPS. The wet well would be sized to accommodate a future plant expansion to 3.0 mgd average max month flow. Space will be allocated in the wet well for future pumps that can be added sequentially as influent flows increase.



AECOM

MISSION SPRINGS WATER DISTRICT
66575 2ND STREET
DESERT HOT SPRINGS, CA 92240

PROCESS FLOW DIAGRAM
SBR ALTERNATIVE

Proj. No.: 60551186		Date:	JUNE 2018
Project: WEST VALLEY WATER RECLAMATION FACILITY		Figure:	6-1

The proposed pumps for the Phase 1 IPS can only be conservatively ramped down to 70% of full speed to achieve a minimum flowrate of approximately 800 gpm per pump. The initial influent flow for the IPS is expected to be approximately 0.2 mgd (170 gpm). It is therefore recommended to install two smaller temporary pumps in a duty and standby configuration to handle the initial flows to the IPS. Preliminary pump selection for the temporary pumps is 3-inch submersible pumps with 5 HP motors, and a VFD. The smaller temporary pumps can be removed and replaced with the larger Phase 1 pumps described above once flows into the IPS begin to increase.

6.2.2 Future Design

To meet a future condition, the IPS would be designed to pump a peak flow of 5,200 gpm (7.50 mgd). Two additional pumps would be installed for the future Phase 2 flows for a total of five pumps. Four pumps would be duty pumps, and one pump would be on standby. Preliminary pump selection is 6-inch submersible pumps with 30 HP motors and VFDs. The two new pumps would have the same characteristics and operating criteria as Phase 1.

6.3 Preliminary Treatment

For preliminary treatment, coarse screens, fine screens, and vortex grit removal technologies were selected as the recommended alternative. This section discusses the design considerations for preliminary treatment. The structure will be designed to hydraulically accept the peak Phase 2 flow condition.

6.3.1 Phase 1 Design

Phase 1 for screening would involve installation of the following equipment. Details of equipment are provided in Table 6-1.

- Multi-rake screen with ¼-inch spacing
- A length of open channel will be reserved downstream of the ¼-inch screens to accommodate installation of rotary drum screens with 1 to 2 mm perforations (fine screens) in the event the SBR plant is converted to MBR in the future

Table 6-1 – Phase 1 Screening Design

Item	Coarse Screen – Multi-rake
Number of Channels	2 (1 duty, 1 standby)
Screen Spacing	1/4-inch
Peak Flow	3.75 mgd
Head loss at 30% Blinding	9 inches

Phase 1 for grit removal would involve installation of two vortex grit chambers each designed to handle peak wet weather flow of 3.75 mgd, providing full redundancy in the system. Flow is directed through the vortex grit chambers continuously, and grit is pumped intermittently from the bottom of the chamber to the grit dewatering unit. The vortex grit chamber has a propeller blade that runs continuously to assist vortex movement of wastewater. Recessed impeller vortex grit pumps were chosen for the design. Details of equipment for the grit removal system are provided in Table 6-2.

Table 6-2 – Phase 1 Grit Removal Design

Item	Grit Removal	Comments
Grit Removal Technology	Vortex Grit Removal	With propeller blade
Number of Units	2	Duty/ standby configuration
Peak Flow	3.75 mgd	Per unit
Chamber Configuration	270 degree chamber with baffle	
Head loss across Unit	5 inches	
Grit Removal Efficiency	95% removal of all grit down to 100 mesh (105 micron)	For all flows up to peak condition
Grit Pump Type	Recessed Impeller Vortex Pump	
Number of Grit Pump Units	2	Duty/ standby configuration
Grit Pump Rating	250 gpm	Per unit
Grit Dewatering Unit	Shafted Screw Conveyor	
Number of Dewatering Units	2	Duty/ standby configuration
Dewatering Unit Rating	250 gpm	Per unit

6.3.2 Future Design

Future design would involve an increase of flows to 3.0 mgd, so an additional duty unit would be put in service for each process, with the overall equipment shown below:

Screening:

- Multi-rake screen with ¼-inch spaced multi-rake (2 duty, 1 standby).

Grit removal:

- Vortex grit removal units (2 duty, 1 standby).
- Recessed impeller grit pumps (2 duty, 1 standby).
- Grit dewatering units (2 duty, 1 standby).

Alternatively for grit removal, the two grit chambers could be retrofitted to hold HeadCell units, which may avoid installation of a third vortex grit removal chamber.

During a workshop, MSWD stated their preference for HeadCell de-gritters. For the purpose of this report, Vortex grit removal units were used as the basis of space planning. The space planning for Vortex units is adequate to support the installation of HeadCell units with no significant changes to structural components. Design details for inclusion of the HeadCell units will be developed during final design.

6.4 Biological Treatment System

The selected biological treatment system is SBR. Provisions will be made in the design for the facility to easily accommodate upgrades to produce recycle water in the future.

6.4.1 Phase 1 Design

Phase 1 will consist of four SBR tanks and associated decant tanks, valves, piping, blowers, instrument controls, fine-pore diffusers, process monitoring sensors, and mixers. The initial turn-down of 0.2 mgd can be effectively and efficiently treated by one or two SBR tanks using proper operational strategies. The other SBR tanks will provide all the necessary backup treatment capacity should something happen that requires a basin to be taken offline. Together with electrical backup power at the facility, the requirements of Class 1 reliability standards are met.

The third and fourth SBRs and decant tanks will be brought online when the average treatment flow reaches approximately 0.7 and 1.0 mgd, respectively. The four SBRs can treat up to 1.5 mgd. The SBRs will be equipped with sufficient aeration capacity to meet the full flow aeration requirement of 8,600 lb/d with one tank out of service to meet Class 1 reliability standards.

Decant from the SBR tanks will flow by gravity through a pipe from the decanter to the associated SBR decant tank. From the decant tanks, a throttling valve will be used to control the clarified effluent discharge for distribution to the percolation ponds.

6.4.2 Future SBR Upgrade to Tertiary Treatment

Space will be reserved as part of the Phase 1 design to accommodate a potential future addition of coagulation, filtration, and disinfection to the SBR process to achieve Disinfected Tertiary Recycled Water standards producing recycle water in accordance with Title 22. Space will also be allocated for a future recycle water storage, and transmission pumping station.

6.4.3 Future Upgrade and Expansion by Conversion to MBR

Space will be allocated as part of the Phase 1 design to accommodate a potential upgrade and expansion in the event that the plant is to be converted from an SBR process to an MBR process in the future. Under this scenario, decant tanks will be converted to membrane tanks. Space will be reserved to add a new Membrane Support Building and UV Disinfection. Space will be reserved to add more channels around the decant and SBR tanks to accommodate an internal recycle flow. Space for additional blowers and connection points on the aeration supply manifold for additional diffusers will also be provided to meet the added aeration demand of a future conversion to MBR. Space will also be allocated for a future recycle water storage, and transmission pumping station.

The SBR basins and decant tanks can be converted one at a time to allow for continued treatment during the conversion process. The firm treatment capacity of 1.5 mgd will not be compromised during the conversion of each SBR tank. To convert an SBR tank, additional blowers and diffusers will be added to meet the new aeration demand. A new wall will be added to the tank to create separate anoxic and aerobic tanks, with sufficient space in the aerobic tank for 0.15 MG of equalization capacity to buffer against peak flows and take the place of the decant tanks. The aeration basin effluent will flow by gravity to the distribution channel for gravity feeding of the membrane tanks and internal recycle channel. Effluent pumps will provide the pressure needed to pull water through the MBR membrane elements and onto the disinfection process.

6.5 Effluent Management

Treatment plant effluent will be disposed by discharging to infiltration basins. The number of basins would increase between initial flows and future flows.

6.5.1 Phase 1 Design

For Phase 1 flows of 1.5 mgd, two infiltration basins and one redundant basin would be used to dispose of treated effluent. Additional basins should be built when needed as plant flows increase to decrease initial capital investment. AECOM's site specific geotechnical investigation measured an infiltration rate of 6-9 in/hr. Based on EPA guidelines, 7-10% of the measured rate should be used for design, thus, the design infiltration rate will be 2 in/hr, Square basins were sized assuming a water depth of one foot in each infiltration basin. A depth of one foot is a "rule of thumb" operational criterion to not allow solids to be driven deep into the sand where they cannot be reduced through plowing/harrowing during maintenance. Operating at higher water depths in the percolation ponds could increase maintenance costs. Infiltration basins will be located in the southern portion of the WVWRF site to allow for sufficient distance from MSWD's Public Water Supply Well 33 to eliminate the infiltration basins' influence on groundwater flowing to Well 33 as demonstrated by groundwater modeling. See Groundwater Model To Evaluate The Potential Impact From The Proposed WVWRF Percolation Basins, EnviroLogic Resources, March 2018, for details. Table 6-3 shows the design parameters of the infiltration basins for Phase 1 Design.

Table 6-3 – Phase 1 Infiltration Basins Design

Parameter	Design Value
Design Infiltration Rate	2.0 feet/day
Number of Basins	2 + 1
Basin Length	220 feet
Basin Width	220 feet
Basin Depth	5 feet
Water Depth	1 foot
Slope	1:4

6.5.2 Future Design

For future flows of 3.0 mgd, the number of onsite infiltration basins would be doubled. The 80-acre offsite disposal location discussed previously may also be considered as an alternate site to handle future effluent discharges for disposal by infiltration basins.

6.6 Solids Storage and Thickening System

For solids storage and thickening, aerated sludge storage tanks with decanters was selected as the preferred alternative for initial low flows. As the flows increase closer to the Phase 1 design condition of 1.5 mgd, consideration is recommended to provide dedicated mechanical thickening equipment using Rotary Drum Thickeners.

6.6.1 Phase 1 Design

Initial operation of the plant starting at 0.2 mgd)would involve installation of the following equipment for solids thickening. Details of equipment are provided in Table 6-4.

- Aerated sludge storage tanks
- Process air system
- Decanter system

Table 6-4 – Initial Solids Storage and Gravity Thickening Design

Item	Solids Storage with Decanter	Comments
Number of Tanks	2	Duty/ duty configuration
Tank Volume	12,480 ft3	Per tank
Tank Side Water Depth (SWD)	16 ft	
Tank Length	30 ft	Per tank
Tank Width	26 ft	Per tank
Number of Tanks	2	Duty/ duty configuration
Tank Volume	12,480 ft3	Per tank
Tank Side Water Depth (SWD)	16 ft	
Tank Length	30 ft	Per tank
Tank Width	26 ft	Per tank
Diffuser Type	Coarse Bubble 9" Disc Diffuser	
Diffuser Material	EPDM Membrane	
Number of Diffusers (Per Tank)	32	
Number of Grids (Per Tank)	2	16 diffusers per grid
Number of Blowers	2	Duty/ standby configuration
Air Flow Rate	1000 scfm	Per blower
Blower Type	Positive displacement	
Blower Power	60 HP	Per blower

For the Phase 1 design flow condition of 1.5 mgd, dedicated mechanical thickening equipment is recommended for thickening raw WAS from the secondary process or aerated sludge storage tank. Details of mechanical thickening equipment are provided in Table 6-5.

- Rotary drum thickeners
- Sludge feed pumps
- Polymer dosing system

In the future, when it comes time to consider ways to decrease BFP run times, adding an RDT is one alternative to adding another BFP. Adding an RDT can help defer a second BFP by reducing BFP run times. The timing of this change will depend on BFP run times that are acceptable to MSWD.

Table 6-5 – Phase 1 Mechanical Thickening Design

Item	Mechanical Thickening	Comments
Mechanical Thickening Type	Rotary drum thickening	
Number of Thickening Units	2	Duty/ standby configuration
RDT Unit Rating	230 gpm	Per unit
Number of Sludge Feed Pumps	2	Duty/ standby configuration
Sludge Feed Pump Type	Progressive cavity	
Sludge Feed Pump Rating	230 gpm	Per pump
Polymer Dosing Pump Type (Neat)	Peristaltic	
Number of Polymer Dosing Pumps	2	Duty/ standby configuration

(Neat)		
Polymer Dosing Pump Rating (Neat)	10 GPH	Per pump
Polymer Dosing Pump Type (Dilution)	Peristaltic	
Number of Polymer Dosing Pumps (Dilution)	2	Duty/ standby configuration
Polymer Dosing Pump Rating (Dilution)	2400 GPH	Per pump

6.6.2 Future Design

Future design would involve an increase of flows to 3.0 mgd, so an additional duty unit would be put in service, with the overall equipment shown below:

- Rotary drum thickeners (2 duty, 1 standby)
- Sludge feed pumps (2 duty, 1 standby)
- Polymer dosing system (1 duty, 1 standby)

6.7 Solids Dewatering System

For solids dewatering, the preferred alternative for Phase 1 design and Future design is a combined GBT and BFP (3-belt BFP). This section discusses the design considerations for the solids dewatering system.

6.7.1 Phase 1 Design

Phase 1 for solids dewatering would involve installation of the following equipment. Based on experience at the Horton WWTP, MSWD has a preference for progressive cavity sludge feed pump by Seepex preceded by an in-line grinder by Muffin Monster. Details of equipment are provided in Table 6-6.

- Combined GBT and BFP (3-Belt BFP)
- Sludge feed pump
- Polymer dosing system

Table 6-6 – Phase 1 Solids Dewatering Design

Criteria	Solids Dewatering	Comments
Dewatering Unit Type	3-Belt BFP	Combination GBT & BFP
Number of Dewatering Units	2	Duty/ standby configuration
Unit Rating	300 gpm	Per unit
Belt Length	2.0 meter	
Number of Sludge Feed Pumps	2	Duty/ standby configuration
Sludge Feed Pump Type	Progressive cavity	
Sludge Feed Pump Rating	300 gpm	Per pump
Polymer Dosing Pump Type (Neat)	Peristaltic	
Number of Polymer Dosing Pumps (Neat)	2	Duty/ standby configuration
Polymer Dosing Pump Rating (Neat)	10 GPH	Per pump
Polymer Dosing Pump Type (Dilution)	Peristaltic	
Number of Polymer Dosing Pumps (Dilution)	2	Duty/ standby configuration

Polymer Dosing Pump Rating (Dilution)	2400 GPH	Per pump

6.7.2 Future Design

Future design would involve an increase of flows to 3.0 mgd, so an additional duty unit would be put in service for the solids dewatering process, with the overall equipment shown below:

- 3-Belt BFP (2 duty, 1 standby)
- Sludge feed pumps (2 duty, 1 standby)
- Polymer dosing system (1 duty, 1 standby)

At a workshop, MSWD indicated preference for progressive cavity pump manufactured by Seepex. They also indicated preference for a Dynablend in-line mixing feed polymer system. These are to match existing equipment at the Horton WWTP.

6.8 Solids Stabilization and Disposal

The preferred alternative for Phase 1 and Future Design for solids stabilization and disposal is aerated sludge storage and dewatering. Dewatered solids will be discharged straight into a trailer and trucked offsite for disposal. No solids stabilization will be undertaken at the plant.

6.8.1 Phase 1 Design

Phase 1 for solids stabilization and disposal would involve installation of the following equipment. Details of equipment are provided in Table 6-7.

- Discharge conveyor
- Load out conveyor

Table 6-7 – Phase 1 Solids Stabilization and Disposal Design

Criteria	Solids Dewatering	Comments
Discharge Conveyor Type	Shafted Screw Conveyor	Inclined
Number of Discharge Conveyors	1	Duty
Discharge Conveyor Size	12” diameter, 30 foot long	
Load Out Conveyor Type	Shafted Screw Conveyor	
Number of Load Out Conveyors	1	Duty
Load Out Conveyor Size	12” diameter, 30 foot long	

6.8.2 Future Design

Future design would involve an increase of flows to 3.0 mgd. A new discharge conveyor will be provided with the additional belt filter press. The truck load-out conveyor will be sized in Phase 1 to accommodate the dewatered solids from the treatment plant at a capacity of 3.0 mgd.

6.9 Odor Control and Air Dispersion Modeling

Odor control is needed at the influent pump station, preliminary treatment areas, and solids processing and storage areas to capture and treat the foul air anticipated to be generated from these facilities.

As part of the design of odor control system, it is necessary to consult the South Coast Air Quality Management District (SCAQMD) to ensure applicable regulations are satisfied. SCAQMD requires zero detection of odors at the property fence line, which may require air dispersion modeling using a screening or detailed model such as AERSCREEN or AERMOD.

6.9.1 Phase 1 Design

For Phase 1, odors will be controlled by a system consisting of covers over selected process areas, foul air collection, and activated carbon treatment before exhausting to atmosphere. Air flow is expected to be 2,300 cfm. The predominant wind direction is from the west. The exhaust from the odor control system will be located near the middle interior of the site to allow for air dispersion before reaching the east property boundary.

6.9.2 Future Design

The design of an odor control system for a future expansion to 3.0 mgd will be revisited based on the performance of the activated carbon system.

6.10 Site Layout and Access Roadways

This section describes the site layout and access roadways for the new WVWRF. The new WVWRF site would have a paved entrance road and parking area. The area surrounding the IPS, headworks, process building, and SBR would also be paved similar to the existing Horton WWTP site. Gravel roads would be provided for access to the effluent spreading basins.

The overall site will be orientated as per the Master Site Plan Conceptual Layout shown in Figure 6-2. The facilities are located to take advantage of the natural ground sloping from north to south for efficient hydraulic designs of treatment processes and drainage. Where possible, facilities are also oriented to provide shelter from predominant strong winds from the west.

The influent pump station will be placed near the future conveyance system located within Little Morongo Road. The plant will be placed to the west of Little Morongo Road by approximately 500 feet, and south of the existing domestic water production well by approximately 1,000 feet. The site layout for the plant utilizes common walls for many unit processes and tanks, allowing for a smaller footprint. The plant will take up an area of approximately 1.5 acres. The infiltration basins will be located in the southern portion of the site to provide adequate offset distance from the domestic water production well at the northeast corner of the site, while also allowing some buffer between the basins and the south property line where monitoring wells will be located. The infiltration basins will take up an area of approximately 7 acres. The primary site access will be from Little Morongo Road.


EXISTING
PRODUCTION
WELL SITE
FUTURE
HEADWORKS
FUTURE
PRIMARY TANKS
MSWD CONVEYANCE SYSTEM
225'
FUTURE
PROCESS TANKS
185'
INFLUENT
PUMP STATION
1.5 MGD
SBR PLANT
FUTURE ANAEROBIC
DIGESTORS
PLANT EFFLUENT
LITTLE MORONGO ROAD
455'
INFILTRATION
BASINS
455'
FUTURE
INFILTRATION
BASINS
20TH AVENUE
0 75' 150' 300'
N
AECOM
MISSION SPRINGS WATER DISTRICT
66575 2ND STREET
DESERT HOT SPRINGS, CA 92240
SITE MASTER PLAN
CONCEPTUAL LAYOUT
Proj. No.: 60551186
Date: JUNE 2018
Project: WEST VALLEY WATER RECLAMATION FACILITY
Figure: 6-2

6.10.1 Phase 1 Design

Phase 1 treatment plant will have a capacity of 1.5 mgd average max month daily flow. The layout will be able to accommodate either an SBR upgrade to disinfected tertiary recycled water or an upgrade and expansion to a 3.0 mgd MBR treatment plant that is also capable of producing disinfected tertiary recycled water.

A preliminary layout for the 1.5 mgd SBR plant is illustrated in Figure 6-3. The preliminary treatment would consist of coarse screens followed by grit removal. A space would be allocated for the addition of fine screens downstream of the ¼-inch travelling rakes (upstream of the grit removal) to accommodate a possible future conversion to an MBR process. The SBR system would consist of four process tanks and four decant tanks. The process and decant tanks would be interconnected though a common distribution channel so that any process tank could discharge to any one decant tank. Sludge generated in the SBR process would be conveyed to aerobic sludge storage tanks with decanters for gravity thickening. Solids dewatering of both secondary and preliminary solids would occur in a dedicated solids handling room. A dedicated chemical storage and delivery room would also be provided to house all the chemicals for the plant. An administration building consisting of a control room, lab, break room, storage room, and men's and women's locker rooms would also be provided. A dedicated electrical room would be provided to house all the electrical panels and equipment for the plant. The process air blowers and emergency generator for the plant would be located outside on a concrete pad or open air buildings for protection from the elements and noise reduction. A canopy structure could be provided over this area, as well as other areas of the plant, to provide protection from the sun and also from blowing debris during strong wind conditions.

6.10.2 Future SBR Upgrade to Tertiary Treatment

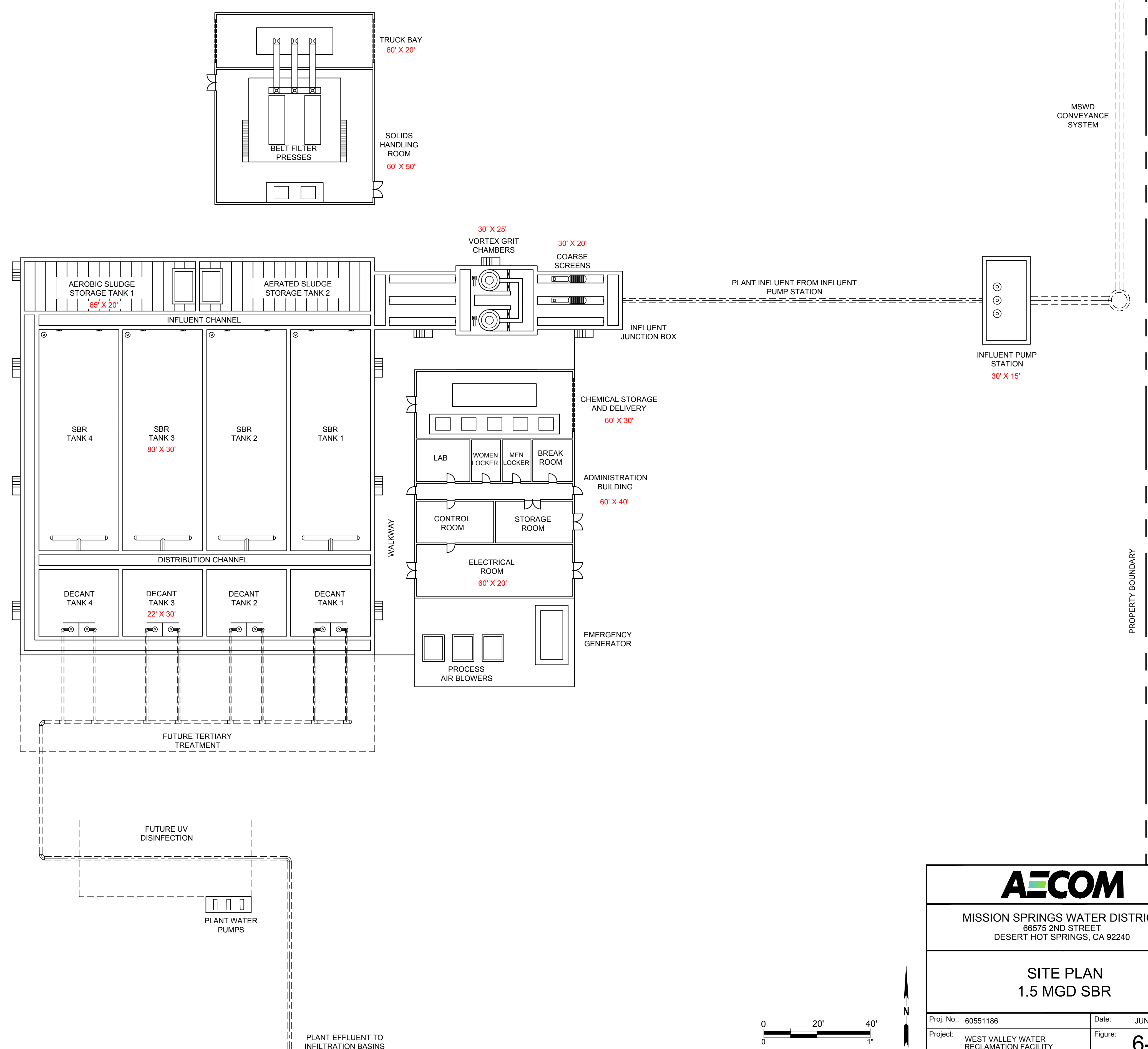
Figure 6-4 shows the Phase 1 site layout with the addition of coagulation, filtration, and disinfection to upgrade the 1.5 mgd SBR treatment plant to achieve Disinfected Tertiary Recycled Water standards producing recycle water in accordance with Title 22. Space is also be allocated for a future recycle water transmission pumping station.

6.10.3 Future Upgrade and Expansion by Conversion to MBR

Figure 6-5 shows a site layout assuming a future upgrade and expansion to 3.0 mgd with the conversion to an MBR treatment plant in the future. Under this scenario, effluent will achieve Disinfected Tertiary Recycled Water standards producing recycle water in accordance with Title 22.

6.11 Site Lighting

All of the outdoor lighting at the site would be provided by light fixtures attached to new structures. A few free standing light poles would be located in areas that do not receive sufficient lighting from the buildings such as the entrance drive from Little Morongo Road and the access road to the spreading basins.

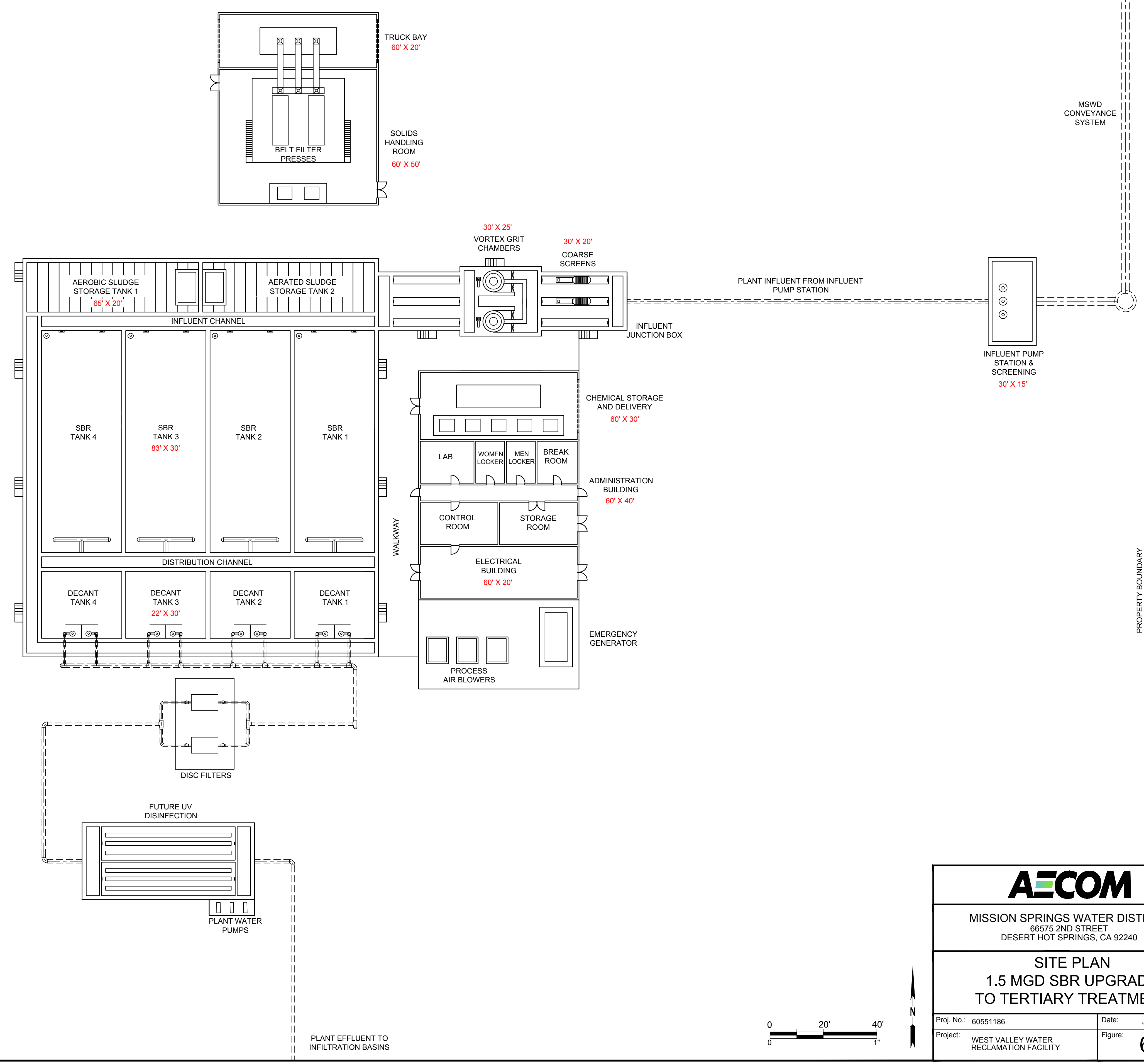


AECOM

MISSION SPRINGS WATER DISTRICT
66575 2ND STREET
DESERT HOT SPRINGS, CA 92240

SITE PLAN
1.5 MGD SBR

Proj. No.: 60551186	Date: JUNE 2018
Project: WEST VALLEY WATER RECLAMATION FACILITY	Figure: 6-3

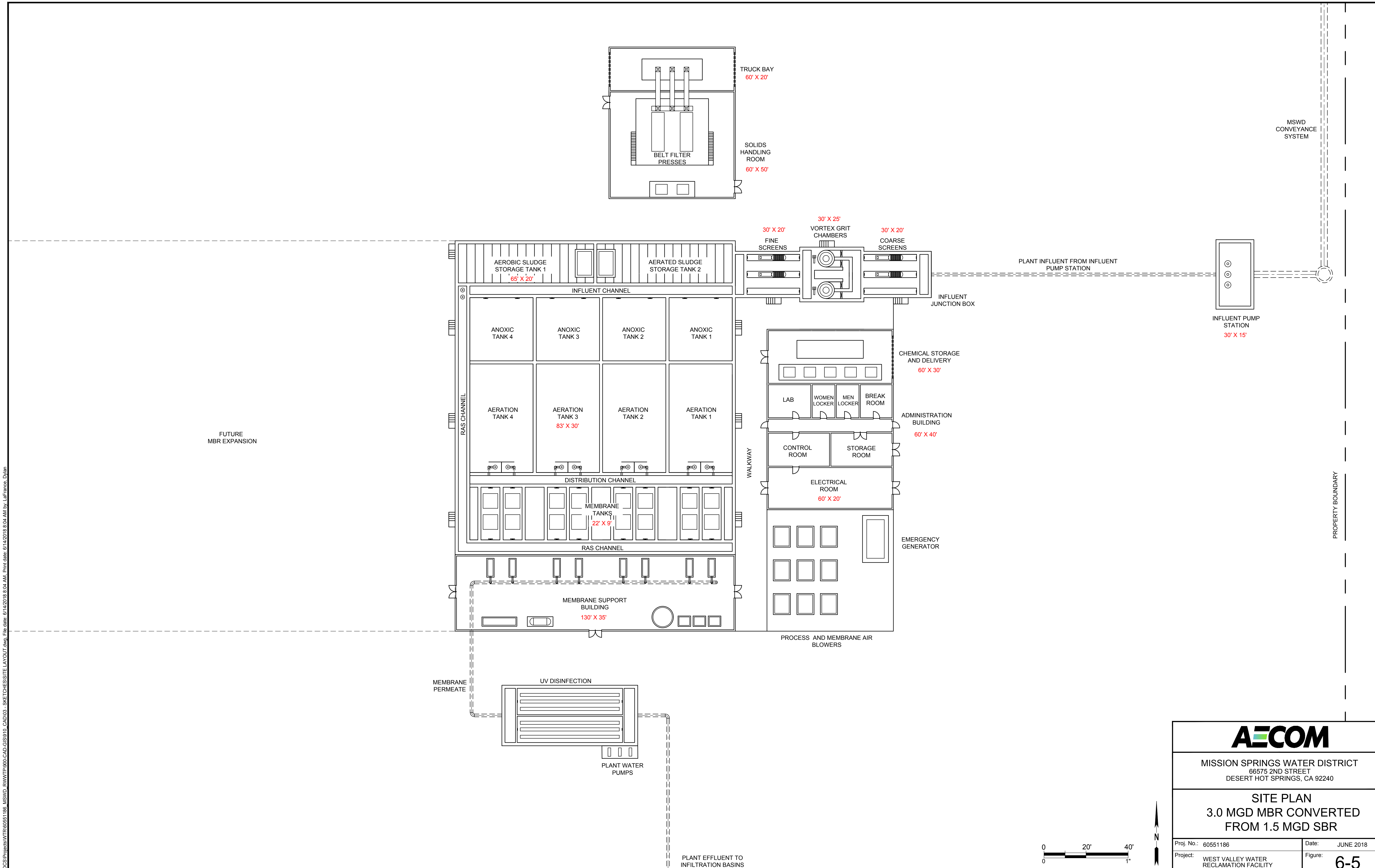


AECOM

MISSION SPRINGS WATER DISTRICT
66575 2ND STREET
DESERT HOT SPRINGS, CA 92240

SITE PLAN
1.5 MGD SBR UPGRADE
TO TERTIARY TREATMENT

Proj. No.:	60551186	Date:	JUNE 2018
Project:	WEST VALLEY WATER RECLAMATION FACILITY	Figure:	6-4


TRUCK BAY
60' X 20'
BELT FILTER PRESSES
SOLIDS HANDLING ROOM
60' X 50'
MSWD CONVEYANCE SYSTEM
30' X 25'
VORTEX GRIT CHAMBERS
30' X 20'
FINE SCREENS
30' X 20'
COARSE SCREENS
PLANT INFLUENT FROM INFLUENT PUMP STATION
INFLUENT JUNCTION BOX
INFLUENT PUMP STATION
30' X 15'
AEROBIC SLUDGE STORAGE TANK 1
65' X 20'
AERATED SLUDGE STORAGE TANK 2
INFLUENT CHANNEL
ANOXIC TANK 4
ANOXIC TANK 3
ANOXIC TANK 2
ANOXIC TANK 1
AERATION TANK 4
AERATION TANK 3
83' X 30'
AERATION TANK 2
AERATION TANK 1
RAS CHANNEL
DISTRIBUTION CHANNEL
MEMBRANE TANKS
22' X 9'
RAS CHANNEL
WALKWAY
CHEMICAL STORAGE AND DELIVERY
60' X 30'
LAB
WOMEN LOCKER
MEN LOCKER
BREAK ROOM
ADMINISTRATION BUILDING
60' X 40'
CONTROL ROOM
STORAGE ROOM
ELECTRICAL ROOM
60' X 20'
EMERGENCY GENERATOR
PROPERTY BOUNDARY
FUTURE MBR EXPANSION
MEMBRANE SUPPORT BUILDING
130' X 35'
PROCESS AND MEMBRANE AIR BLOWERS
MEMBRANE PERMEATE
UV DISINFECTION
PLANT WATER PUMPS
PLANT EFFLUENT TO INFILTRATION BASINS
0
20'
40'
0
1"
N
AECOM
MISSION SPRINGS WATER DISTRICT
66575 2ND STREET
DESERT HOT SPRINGS, CA 92240
SITE PLAN
3.0 MGD MBR CONVERTED FROM 1.5 MGD SBR
Proj. No.: 60551186
Date: JUNE 2018
Project: WEST VALLEY WATER RECLAMATION FACILITY
Figure: 6-5

6.12 Stormwater Control

The site grading would be designed to provide gentle site slopes and swales to allow localized stormwater infiltration. Retention basins will be provided in selected locations to drain excess water away from full swales during extreme rainstorms. The excess stormwater would be directed to the spreading basins at the WVWRF site via storm drain piping. Site plans will include provisions to allow stormwater run-on to pass through and/or around the site.

6.13 Landscaping and Irrigation

Areas that are not covered with asphalt or gravel pavement would be covered with native soils.

Fencing and drought tolerant landscaping would be provided in selected locations. Use of irrigation would be minimized.

Section 7 Design Criteria

7.1 Process Design

The process design criteria for the new WVWRF are listed in Table 7-1. The WVWRF design includes the following process facilities:

- IPS to provide a Phase 1 peak hour capacity of 3.75 mgd.
- Preliminary treatment consisting of: two (2) coarse screens, two (2) vortex grit chambers, two (2) grit pumps, grit dewatering, two (2) fine screens, screenings dewatering.
- Four (4) new SBRs for treatment of BOD_5, TSS, and TN.
- Infiltration basins for effluent disposal
- Two (2) aerated sludge storage tanks and decanters.
- Provide space for two future rotary drum thickening system (for flows above 1.5 MGD)
- Solids dewatering building housing two (2) combination gravity belt thickeners/belt filter presses (3-Belt BFP)s and dewatered biosolids out loading.
- An odor control system using activated carbon treating foul air exhausted from the IPS wet well, headworks, SBR tanks, solids dewatering, solids storage bins.

Table 7-1 – Process Design Criteria

Item and Description	WVWRF Design Criteria
WASTEWATER CHARACTERISTICS	
Average Day Max Month (ADMM) flow, mgd	1.5
Peak Hour Flow, mgd	3.75
Maximum Monthly BOD5, mg/L	330
Maximum Monthly BOD, lb/day	4,541
Maximum Monthly TSS, mg/L	370
Maximum Monthly TSS, lb/day	5,092
Maximum Monthly NH_4-N, mg/L	43
Maximum Monthly NH_4-N, lb/day	592
Maximum Monthly TKN, mg/L	60
Maximum Monthly TKN, lb/day	826
Effluent BOD5, mg/L	30

Item and Description	WVWRF Design Criteria
Effluent TSS, mg/L	30
Effluent NO_3-N, mg/L	<10
Effluent Total Nitrogen, mg/L	10
INFLUENT PUMP STATION	
Type of Pump	Submersible
Number of Large Pumps	3 (2 duty, 1 standby)
Motor Size Each (HP)	30
Drive Type	VFD
Capacity of Each Pump (gpm)	1,300
Total Dynamic Head – TDH (ft.)	35
Minimum Pump Efficiency at Duty Point (%)	68
Maximum Pump Speed (rpm)	1,800
INFLUENT PUMP STATION WET WELL	
Number of Wet Wells	1
Wet Well Length (ft)	20
Wet Well Width (ft)	10
Wet Well Depth (ft)	18
Pump Operating Range (ft)	5
Minimum Pump Submergence (ft)	8
Pump Cycles (No./hr)	2.6
INFLUENT METERING	
Type	Magnetic
Number	1
Size (inches)	12
Velocity at Peak Flow (fps)	7.4
SCREENING	
Type	Multi-rake
Number	2 (1 duty, 1 standby)
Perforation Size	¼-inch
Maximum Capacity (gpm)	2,600
GRIT REMOVAL AND GRIT HANDLING	
Type	Vortex grit chambers
Number	2 (1 duty, 1 standby)
Peak Capacity, each (mgd)	3.75
Type of Grit Pump	Recessed Impeller Vortex Pump
Number of Grit Pumps	2 (1 duty, 1 standby)

Item and Description	WVWRF Design Criteria
Grit Pump Capacity, each (gpm)	250
Type of Grit Dewatering	Shafted Screw Conveyor
Number of Grit Dewatering Units	2 (1 duty, 1 standby)
Grit Dewatering Unit Capacity, each (gpm)	250
BIOREACTORS (SBR)	
Average Day Max Month (mgd)	1.5
Peak Day Flow (mgd)	2.4
Peak Wet Weather Flow (mgd)	3.75
Maximum Water Temperature (°C)	30
Minimum Water Temperature (°C)	22
Number of Tanks	4 (3 + 1 standby)
Tank Length/Width Ratio	3:1
Tank Length (ft)	83
Tank Width (ft)	30
Top Water Level (ft)	20
Bottom Water Level (ft)	12
TWL at Design Average Flow (ft)	20
Required Total SBR Volume (mg)	1.13
Design Aerobic SRT (days)	5
Design System Total SRT (days)	10
Target Effluent Water Quality (BOD/TSS/TN) (mg/L)	30/30/10
Design MLSS @ TWL (mg/L)	3,733
Design MLSS @ BWL (mg/L)	5,600
WAS PUMPS (SBR)	
WAS Pump Type	Submersible Centrifugal
Number of WAS Pumps	4
WAS Pump Rate (gpm)	300
WAS Pump Discharge Head (ft)	15
WAS Pump Motor Size (HP)	3
MIXERS (SBR)	
Mixer Type	Submersible
Number of Mixers (per basin)	2 (plus a shelf spare)
Mixer Motor Size, each (HP)	15
AERATION SYSTEM (SBR)	
Aeration Type	Diffused air, fine bubble
Design SOR (lb O_2/d)	13,044

Item and Description	WVWRF Design Criteria
Total Aeration Time / Cycle (hr/cycle)	3.0 (4 Cycles)
Oxygen Transfer Efficiency (ADF) (%)	31.7
Blowers Design Air Flow (scfm)	1,105
Total Aeration Time / Cycle (hr/cycle)	3.0
Blowers Type	Positive Displacement with VFD for variable loads and depths
Number of Blowers	4
Total Max Discharge Pressure (psig)	9.11
Total Min Discharge Pressure (psig)	6.51
Blowers Speed (rpm)	1800
Blowers Motor Size (HP)	75
DECANT TANK (SBR)	
Number of Tanks	4
Type	Concrete
Storage Requirement at Average Flow (hr)	3.8
Decant Tank Width (ft)	30
Decant Tank Length (ft)	22
Operating Depth (ft)	12
INFILTRATION BASINS	
Number of Basins	3 (2 duty, 1 standby)
Design Infiltration Rate (ft/d)	3.0
Wetted Basin Floor Area (ft^2)	48,400
Basin Length (ft)	220
Basin Width (ft)	220
Width to Length Ratio	1:1
Basin Depth (ft)	5
Water Depth (ft)	1
Basin Side Slope	1:4
SOLIDS STORAGE TANK	
Type	Aerated Storage Tank
Number	2 (duty / duty)
Tank Volume (ft^3)	12,480
Tank Side Water Depth (ft)	16
Tank Length (ft)	30
Tank Width (ft)	26
Type of Aeration	Coarse Bubble

Item and Description	WVWRF Design Criteria
Diffuser Type	EPDM Membrane Disc Diffuser (9")
Air Blower (each)	2 (1 duty, 1 standby)
Air Flow Rate (scfm)	1,000
Blower Size, each (HP)	60
MECHANICAL THICKENING	
Type of Unit	Rotating Drum Thickener (RDT)
Number of Units	2 (1 duty, 1 standby)
Solids Feed Concentration (mg/L)	7,500
Average WAS Flow (gpd)	6,800 (initial)
Peak WAS Flow (gpd)	140,600
Sludge Flow each Unit (gpm)	230
Peak Solids Loading Rate (lb/hr)	1730
Avg Operating Time, All Units (hr/wk)	35
Peak Operating Time, All Units (hr/wk)	40
Minimum Thickened Sludge Concentration (% TS)	4
Minimum Filtrate Capture (% TS Removed)	95
Polymer Type	Liquid
Polymer Dose (lb/dry ton)	15
Peak Polymer Use (lb/hr)	2
Active Polymer Concentration (%)	30
Polymer Solution Concentration (%)	0.30
Peak Polymer Solution Flow (gph)	900
SOLIDS DEWATERING	
Type of Unit	3-Belt BFP
Number of Units	2 (1 duty, 1 standby)
Size of Unit (meters)	2.0
Annual Average Solids Production (lb/day)	430 (initial)
Peak Month Solids Production (lb/day)	8900
Solids Feed Concentration (mg/L)	7,500 (initial)
Sludge Flow each Unit (gpm)	300
Avg Operating Time, All Units (hr/wk)	3 (initial)
Peak Operating Time, All Units (hr/wk)	55
Minimum Dewatered Cake Solids (% TS)	16
Minimum Filtrate Capture (% TS Removed)	95
Polymer Type	Liquid

Item and Description	WVWRF Design Criteria
Polymer Dose (lb/dry ton)	30
Peak Polymer Use (lb/hr)	7
Active Polymer Concentration (%)	30
Polymer Solution Concentration (%)	0.30
Peak Polymer Solution Flow (gph)	1,000
ODOR CONTROL	
Type	Activated Carbon
Foul Air Flow (cfm)	10,000
Influent H_2S (ppm)	60
Discharge H_2S Limit (ppm)	1.0

7.2 Site and Civil Design

Civil and site work for WVWRF construction would include grading, drainage, and site improvements. The area around new structures would be backfilled to match existing contours where feasible. Structures that extend above the surrounding grade would be backfilled to protect the structure from weathering. In general, slopes that are not subject to regular traffic would be graded to a maximum 4:1 side slope. Slopes that are subject to regular traffic would be graded to a maximum 6:1 side slope. All disturbed areas would be paved, or covered with native soil..

Areas that require routine vehicle access would be bituminous concrete roadways, consisting of a 12-inch gravel base course, a 2½-inch bituminous concrete binder course and a 1½-inch bituminous concrete top course. Areas that require routine pedestrian access would have concrete sidewalks. The sidewalk would consist of 4 inches of reinforced concrete on an 8-inch gravel base course.

Painted steel bollards (approximately 4 inches in diameter and 42 inches high) would be provided as needed to protect equipment or structures that are near roadways. Landscaping and planting at the site would blend into the existing surrounding conditions to the extent possible.

The site is located outside the 100-year flood plain and there are no apparent wetlands on site which collectively exempts the site from USACE permitting. The site is located inside the 500-year flood plain as shown in Figure 7-1. The site lies within a larger alluvial fan area which makes it subject to potential flash floods. Requirements for protecting the site from flash floods will be included for Plan Check approval including provisions to route stormwater run-on to pass through or around the site.

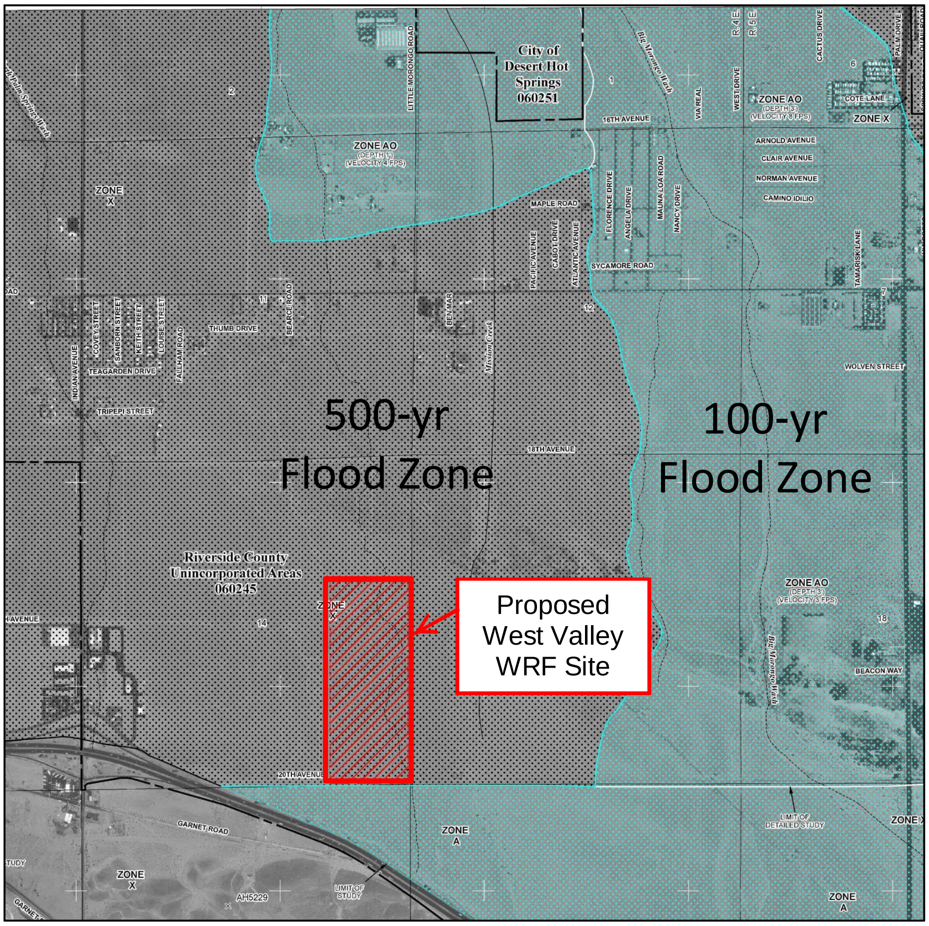


Figure 7-1 – FEMA Flood Map showing location of Proposed Project Site

7.3 Geotechnical Design

A geotechnical investigation was performed by AECOM. A copy of the Geotechnical Investigation Report is provided as Appendix A.

Subsurface soils encountered during the field exploration consist of medium dense to very dense silty sands and poorly graded sands with silts and gravel. The surficial soils are loose in nature (possibly soils deposited due to winds over several years).

San Andreas Fault (San Bernardino South segment) which is a strike-slip fault with a maximum moment magnitude of 7.9 is identified on the north end of the project site. Peak ground acceleration anticipated at the project site is estimated to be on the order of 0.88g which is based on a 5% probability of exceedance in 50 years (975-year return period).

The geological, geotechnical and seismic conditions at the project site have been evaluated in terms of their impact on the planned project. Based on the findings of the investigation, it is AECOM's opinion that the planned project is feasible provided the recommendations of the geotechnical report are implemented in design, constructions, and operations/maintenance of the project. It is anticipated that geologic hazards and geotechnical concerns will not significantly impact the project.

7.3.1 Foundations and Excavation Considerations

Shallow foundations are a proposed option for the project site. It is anticipated that all structures will be founded on mat foundations or slabs-on-grade. The bearing capacity of these foundations will be limited by settlement. A maximum allowable bearing pressure of 2,000 per square foot (psf) is recommended.

Clearing and grubbing would be necessary for the project site to be clear of any existing vegetation, rubbish, spongy, hazardous, deleterious materials and debris.

Due to the presence of loose soils at the anticipated bottom of footing elevation, it is recommended that soils within 3 feet from the bottom of foundation or slab on grade be removed and replaced with structural backfill following recommendations in the geotechnical report. The compacted fill should extend a minimum of 5 feet beyond the edges of foundation.

Majority of the project site soils are suitable for use as structural fill provided it does not contain rocks or hard lumps greater than 3 inches in maximum dimension and shall have at least 80% passing the ¾-inch sieve, at least 25% passing No. 4 sieve and less than 5% passing the No. 200 sieve. It is noted that backfill material such as pea gravel and crushed rock do not meet the requirements for structural fill due to their relatively high permeability and thereby provide the potential to collect water.

Excavations during construction should be performed in accordance with applicable local, state, and federal regulations including the current California Occupational Safety and Health Administration (Cal/OSHA) excavation and trench safety standards. Unsupported temporary slopes with conditions similar to those encountered during the exploration (Cal/OSHA Type C soils) should be made at an inclination no steeper than 1.5:1 (horizontal to vertical), or flatter if field conditions so dictate.

As soil conditions may vary, the contractor should employ an excavation competent person as defined by Cal/OSHA to determine all aspects of excavation safety. The design and construction of temporary excavation support systems (e.g., shoring) and temporary slopes, as well as the maintenance and monitoring of these works during construction, is the responsibility of the contractor. Trench excavations should be made with nearly vertical sides, using sheeting and shoring whenever required. Soils encountered during our field investigation should be rippable with conventional earthwork equipment.

7.3.2 Groundwater Considerations

Groundwater was not encountered in any of the borings performed for this project to a maximum depth of 50 feet below ground surface (bgs). Based on the review of historic groundwater records in the project area, the groundwater levels are deeper than 180 feet below ground surface (bgs). Groundwater level measurements from the production well (MSWD Well 33) on the north end of the project site shows levels deeper than 150 feet bgs.

7.4 Structural Design

All reinforced concrete structures would be designed in accordance with the latest editions of the American Concrete Institute (ACI) 318, ACI 350 and the California Building Code. Structural concrete would be 4,000 pounds per square inch (psi) compressive strength using Type II cement conforming to American Society for Testing and Materials (ASTM) C150 and aggregate conforming to ASTM C33. Reinforced steel would be Grade 60 in accordance with ASTM A615.

In addition to process equipment loads, structure loadings will be developed from internal static and earthquake dynamic pressures and exterior earth pressures per the project geotechnical report.

Water stops would be provided at all construction joints in hydraulic structures. Water stops would be made from PVC material. The exposed concrete to one foot below the high water line in the IPS wetwell, Headworks and MBR Tanks will be coated with a mechanically-attached (T-lock) PVC liner.

7.5 Architectural Design

The construction of the WVWRF requires new buildings for the IPS and Process Building. The process building includes sludge pumps, blowers, chemical storage and feed equipment and Solids Dewatering, Laboratory, men's and women's locker rooms, break room, control room, storage room, and electrical room. These would be constructed of tan split face reinforced masonry to match existing buildings at MSWD. The roof would be precast concrete plank with rigid insulation and a membrane roofing material installed on the top surface of the insulation.

The doors would be aluminum with aluminum frames. Door hardware would be stainless steel. Windows would be aluminum with double pane insulating glass. Louvers would be aluminum with a colored Kynar protective coating. Handrail would be anodized aluminum.

Grating for stairs, access platforms and sump covers would be aluminum. Tank and channel covers would be solid aluminum or FRP material for corrosion resistance.

Table 7-2 shows a room finish schedule for the new buildings for the WVWRF design.

Table 7-2 – Preliminary Architectural Finish Schedules – Full Design

Description	Item						
	Floor	Base	North Wall	South Wall	East Wall	West Wall	Ceiling
1. IPS Electrical Building							
A. Electrical Room	CS	P	P	P	P	P	P
2. Process Building							
A. Chemical Storage and Delivery	E	E	P	P	P	P	P
B. Solids Handling Room	CS	P	P	P	P	P	P
C. Lab	VT	V	P	P	P	P	AC
D. Women's Locker	CT	CT	P	P	P	P	AC
E. Men's Locker	CT	CT	P	P	P	P	AC
F. Meeting Room/Kitchen	VT	V	P	P	P	P	AC
G. Control Room	VT	V	P	P	P	P	AC
H. Storage	CS	P	P	P	P	P	P
I. Electrical Room	CS	P	P	P	P	P	P
3. Membrane Support Building							
A. Pump/Blower Room	CS	P	P	P	P	P	P

Notes:
CS = Concrete with Sealer
P = Paint
V = Vinyl
VT = Vinyl Tile
CT = Ceramic Tile
E = Epoxy Flooring
AC = Acoustical Ceiling

7.6 Heating Ventilating Air Conditioning and Plumbing Design

7.6.1 Code Standards and References

The WVWRF would have heating, ventilation, air conditioning (HVAC) and plumbing facilities to meet all local, state and national building and plumbing codes. The HVAC and plumbing systems would be designed based on the latest edition of the following standards and codes:

1. American Society of Heating, Refrigeration and Air Conditioning Engineers, Inc. (ASHRAE) Handbooks and Publications
2. National Fire Protection Association (NFPA)
3. Sheet Metal and Air Conditioning Contractor's National Association (SMACNA) Standards
4. California Building Code (CBC)
5. California Mechanical Code (CMC)
6. California Plumbing Code (CPC)

7.6.2 Design Conditions

7.6.2.1 Outdoor Design Temperatures

Summer: 111.2°F. dry bulb, 71.2°F. wet bulb in accordance with ASHRAE standards
Winter: 41.4°F. in accordance with ASHRAE standards

7.6.2.2 Indoor Design Temperatures

Location	Summer Max. Temp. (Deg. F.)	Winter Min. Temp. (Deg. F.)
Electrical Room	85	55
Office Area	75	72
Pump Room	104	55
Chemical Storage Areas	104	55
Truck Bay / Dewatering	104	55

Ventilation Rates:

Location:	Ventilation:
Electrical Room	Air Conditioned
Office Area	15 cfm / person
Chemical Storage / Dewatering	6 air changes per hour (ACH) Summer, 3 ACH Winter
Truck Bay	Odor Controlled, 12 ACH
Digesters	Covered and Odor Controlled, 1 cfm/ft^2 plus added air volume

7.7 Electrical Design

Electrical power for the WVWRF is obtained from the Southern California Edison Company system. The new WVWRF is provided with standby power from a diesel fuel powered engine generator with a nominal capacity rating of 2750 kilowatts, 480-volt, 3-phase power. The standby generator is located outside the new electrical room on a concrete pad. Space was provided in the main switchboard 2500A and for a future MCC with an 600 amp rating.

Table 7-3 shows a motor list for the WVWRF for SBR design.

The conduit for the expansion project would be rigid steel, PVC coated steel, or Schedule 80 PVC depending upon the application and the environmental conditions present within the area. PVC or PVC-coated steel conduit would be used in all wet process areas, and areas where corrosive chemicals are present. In general all outdoor enclosures and enclosures in wet or damp process areas would be National Electrical Manufacturing Association (NEMA) 4X rated constructed using Type 316 stainless steel. Enclosures in separate electrical, control, and office areas would be standard NEMA 1 rated constructed using coated steel.

Table 7-3 – Preliminary Electrical Motor List Process (SBR)

Process Area	WVWRF Equipment	Motor Size (HP)	Motor Starter
Headworks	Influent Pump No. 1	30	VFD
Headworks	Influent Pump No. 2	30	VFD
Headworks	Influent Pump No. 3	30	VFD
Headworks	Fine Screen No. 1	1	Constant Speed-Cross the Line
Headworks	Fine Screen No. 2	1	Constant Speed-Cross the Line
Headworks	Vortex Grit Chamber No. 1	1	Constant Speed-Cross the Line
Headworks	Vortex Grit Chamber No. 2	1	Constant Speed-Cross the Line
Headworks	Grit Pump No. 1	10	Constant Speed-Cross the Line
Headworks	Grit Pump No. 2	10	Constant Speed-Cross the Line
Headworks	Grit Dewatering Unit No. 1	1	Constant Speed-Cross the Line
Headworks	Grit Dewatering Unit No. 1	1	Constant Speed-Cross the Line
SBR Treatment	SBR Blower No. 1	75	VFD
SBR Treatment	SBR Blower No. 2	75	VFD
SBR Treatment	SBR Blower No. 3	75	VFD
SBR Treatment	SBR Blower No. 4	75	VFD
SBR Treatment	SBR Mixer No. 1	15	Constant Speed-Cross the Line
SBR Treatment	SBR Mixer No. 2	15	Constant Speed-Cross the Line
SBR Treatment	SBR Mixer No. 3	15	Constant Speed-Cross the Line
SBR Treatment	SBR Mixer No. 4	15	Constant Speed-Cross the Line
SBR Treatment	SBR Mixer No. 5	15	Constant Speed-Cross the Line
SBR Treatment	SBR Mixer No. 6	15	Constant Speed-Cross the Line
SBR Treatment	SBR Mixer No. 7	15	Constant Speed-Cross the Line
SBR Treatment	SBR Mixer No. 8	15	Constant Speed-Cross the Line
SBR Treatment	SBR WAS Pump No. 1	3	VFD
SBR Treatment	SBR WAS Pump No. 2	3	VFD
SBR Treatment	SBR WAS Pump No. 3	3	VFD

Process Area	WVWRF Equipment	Motor Size (HP)	Motor Starter
SBR Treatment	SBR WAS Pump No. 4	3	VFD
SBR Treatment	SBR Effluent Pump No. 1	20	VFD
SBR Treatment	SBR Effluent Pump No. 2	20	VFD
SBR Treatment	SBR Effluent Pump No. 3	20	VFD
SBR Treatment	SBR Effluent Pump No. 4	20	VFD
SBR Treatment	SBR Effluent Pump No. 5	20	VFD
SBR Treatment	SBR Effluent Pump No. 6	20	VFD
SBR Treatment	SBR Effluent Pump No. 7	20	VFD
SBR Treatment	SBR Effluent Pump No. 8	20	VFD
Sludge Thickening	RDT Unit No. 1	3	VFD
Sludge Thickening	RDT Unit No. 2	3	VFD
Sludge Thickening	RDT Flocculation Mixer No. 1	1.5	Constant Speed-Cross the Line
Sludge Thickening	RDT Flocculation Mixer No. 2	1.5	Constant Speed-Cross the Line
Sludge Thickening	RDT Feed Sludge Pump No. 1	15	VFD
Sludge Thickening	RDT Feed Sludge Pump No. 2	15	VFD
Sludge Storage	Sludge Storage Blower No. 1	60	VFD
Sludge Storage	Sludge Storage Blower No. 2	60	VFD
Sludge Storage	Blower Ventilation Fan No. 1	5	Constant Speed-Cross the Line
Sludge Storage	Blower Ventilation Fan No. 2	5	Constant Speed-Cross the Line
Sludge Dewatering	BFP Feed Box No. 1	1/3	Constant Speed-Cross the Line
Sludge Dewatering	BFP Feed Box No. 2	1/3	Constant Speed-Cross the Line
Sludge Dewatering	BFP Hydraulic Power Unit No. 1	2	Constant Speed-Cross the Line
Sludge Dewatering	BFP Hydraulic Power Unit No. 2	2	Constant Speed-Cross the Line
Sludge Dewatering	BFP Thickener Drive No. 1	3	VFD
Sludge Dewatering	BFP Thickener Drive No. 2	3	VFD
Sludge Dewatering	BFP Press Drive No. 1a	3	VFD
Sludge Dewatering	BFP Press Drive No. 1b	3	VFD
Sludge Dewatering	BFP Press Drive No. 2a	3	VFD
Sludge Dewatering	BFP Press Drive No. 2b	3	VFD
Sludge Dewatering	BFP Wash Water Pump No. 1	15	VFD
Sludge Dewatering	BFP Wash Water Pump No. 2	15	VFD
Sludge Dewatering	BFP Feed Sludge Pump No. 1	20	VFD
Sludge Dewatering	BFP Feed Sludge Pump No. 2	20	VFD
Sludge Dewatering	BFP Feed Sludge Grinder No. 1	5	VFD
Sludge Dewatering	BFP Feed Sludge Grinder No. 2	5	VFD
Sludge Handling	Discharge Conveyor No. 1	7.5	Across the Line/ Direct Online
Sludge Handling	Discharge Conveyor No. 2	7.5	Across the Line/ Direct Online
Sludge Handling	Load Out Conveyor No. 1	7.5	Across the Line/ Direct Online
Sludge Handling	Load Out Conveyor No. 2	7.5	Across the Line/ Direct Online
Sludge Handling	Belt Conveyor No. 1	5	Across the Line/ Direct Online
Sludge Handling	Belt Conveyor No. 2	5	Across the Line/ Direct Online
Odor Control	Odor Control Unit & Exhaust Fan	25	Constant Speed-Cross the Line

All wiring between buildings or structures would be installed within conduits that are encased in reinforced concrete. In general conduits within the pump rooms or process areas would be exposed to view. Use of imbedded conduits would be avoided. All power wiring would be American Wire Gauge (AWG) copper, sized for the required loads.

Indoor lighting in pump rooms would be provided by energy efficient LED fixtures. Explosion proof fixtures and controls would be used in the IPS wetwell and Headworks areas in accordance with NFPA 820 Regulations.

A limited amount of additional exterior lighting is anticipated for the Headworks, SBR process tank area, and the entrance driveway. Otherwise all other light fixtures would be attached to new structures. The use of free standing light poles would be avoided.

The design would comply with the NEMA, Underwriter Laboratories (UL), National Electric Code (NEC) and the CBC Section 8-904 Requirements.

7.7.1 Electrical Service

The electrical service will include one pad mounted electrical transformer. The utility requirements will be per utility company standards.

7.7.2 Main Switchboard

The power to the station will be 480/277 volts, 3-phase, 4-wire with double ended main tie main configuration. This is the most common bus arrangement for treatment plants. The main switchboard will consist of a pull section and one section with both the utility meter and main circuit breaker at one end. The other end of the switchboard will consist of a main breaker fed from a stand-by generator. The outgoing feeders will be evenly distributed on the sections. The main electrical service will have the capability of powering the electrical equipment in all of the different processes.

7.7.3 Motor Control Center

The MCC will be 480 volts, 3 phase, 3 wire and will include the motor starters. It will be also include a 120/208-volt step-down transformer and panelboard for lighting and auxiliary loads.

7.7.4 Emergency Power

A standby emergency generator will be provided to allow continued operation during a power outage. The following features (with motor loads) will be supported:

- Pump station, power to daily continuous operations is essential. Intermittent operations such as sludge dewatering are not essential for emergency power..
- Exhaust fans.
- Miscellaneous electrical loads.

The fuel tank will be sized for an 8- hour run time during a power outage. The above-grade fuel tank will be double-walled with leak detection alarms. The leak detection alarm will have local and remote annunciation. The remote annunciation will be via the site programmable logic controller (PLC) to Central Control via the supervisory control and data acquisition (SCADA) system.

7.7.5 Building Interior Lighting

Interior lighting will consist of surface-mounted light-emitting diode (LED), 1-foot by 4-foot enclosed and gasketed fixtures. Lighting fixtures shall be California Title 24 compliant. Lighting fixtures with emergency battery packs will be used to provide egress lighting during the transition from normal power to emergency generator power.

7.7.6 Site Exterior (Security) Lighting

Exterior site lighting (security lighting) will consist of LED wall packs mounted on the exterior walls and wall mounted LED fixtures. The exterior lighting will be controlled via photocell controls to turn on at dusk and turn off at dawn. The site lighting levels shall be kept within the property.

7.8 Instrumentation and Controls Design

The instrumentation and controls for the facility are located in Control Panel CP-A, housed within the new electrical room.

New status, alarms, and instrument signals would be transmitted to the main control panel.

Nine (9) new control panels would be provided in the WVWRF expansion:

- CP-B serving the IPS;
- CP-C Serving the Coarse Screening System;
- CP-D Serving the Grit Removal System;
- CP-E serving the SBR System;
- CP-G serving the Solids Dewatering System;
- CP-H service the Solids Storage System;
- CP-I serving the Polymer Feed System; AND
- CP-J Odor Control System.

It is expected all the CP's would be provided by the manufacturer of the equipment served.

Table 7-4 shows a preliminary list of status indication points for the WVWRF design. Table 7-5 shows a preliminary list of alarms for the WVWRF design. Table 7-6 shows a preliminary list of instruments for the WVWRF design.

Table 7-4 – Preliminary Status and Indication Points

WVWRF Equipment
Influent Pump Nos. 1 to 3 VFD
Course Screen and Compactor System Nos. 1 and 2
Grit Pumps No. 1 and 2
Grit Chambers No. 1 and 2
SBR System No. 1 to 4
BFP Systems No. 1 and 2
BFP Transfer Conveyor
Solids Storage Loadout Conveyor

WVWRF Equipment
Odor Control System

Table 7-5 – Preliminary List of Alarms

WVWRF Alarm Function
IPS High Wetwell Level
IPS Low Wetwell Level
Influent Pump No. 1 VFD Fault
Influent Pump No. 2 VFD Fault
Influent Pump No. 3 VFD Fault
Influent Pump No. 1 Thermal Overload
Influent Pump No. 2 Thermal Overload
Influent Pump No. 3 Thermal Overload
Influent Pump No. 1 Moisture
Influent Pump No. 2 Moisture
Influent Pump No. 3 Moisture
Coarse Screen and Compactor System No. 1 Failure
Coarse Screen and Compactor System No. 2 Failure
Grit Pump No. 1 Loss of Seal Water
Grit Chamber No. 1 Overtorque
SBR System No. 1 Failure
SBR System No. 2 Failure
SBR System No. 3 Failure
SBR System No. 4 Failure
BFP System Failure
Solids Storage Bin High Level
BFP Transfer Conveyor Overtorque
Solids Storage Bin High Level
Polymer Feed System Failure
Odor Control System Failure

Table 7-6 – Preliminary List of Instruments

WVWRF Instrument and Function
IPS Wetwell Level
Influent Magnetic Flow Meter
Fine Screen Inlet Water Level
SBR System No. 1 Dissolved Oxygen (DO) Meter
SBR System No. 2 DO Meter
SBR System No. 3 DO Meter
SBR System No. 4 DO Meter
Effluent Parshall Flume Flow Meter

WVWRF Instrument and Function
SBR Waste Activated Sludge Magnetic Flow Meter
BFP Magnetic Flow Meter
Solids Storage Bin Level

7.9 CEQA Coordination

All CEQA work will be provided by a consultant hired by MSWD who will provide the following information to the agency. For the purposes of CEQA, the project can be described as follows:

MSWD is in the preliminary engineering phase for a Regional Wastewater Program (RWP). The RWP consists of three components: construction of the WVWRF, construction of a trunk sewer main to connect existing sewered areas to the WVWRF, and construction of a collection system. MSWD is prepared to begin the process of securing approvals for the proposed WVWRF. Additional information related to the RWP, more specifically the WVWRF, is presented below.

MSWD's mission is to provide, protect, and preserve our most valuable resource, water. In a 1996 study, MSWD learned that the migration of wastewater discharged from septic tanks had and would continue to negatively affect regional groundwater quality.[2] *Additionally, the study concluded that wastewater discharged from the thousands of individual septic systems that lie above the Mission Creek and Desert Hot Springs sub basins, poses a significant threat to the public groundwater resources, and recommended the abatement of these individual wastewater disposal systems.*

In response, the MSWD initiated the GQPP. The program involves constructing municipal wastewater collection and treatment systems that will eliminate individual septic systems that overlie the Mission Creek and Desert Hot Springs sub basins. The Desert Hot Springs communities showed their support for protecting and preserving local groundwater supplies by passing two special assessment districts to aid in funding the GWPP.

MSWD's successful completion of GWPP projects since 2006, and continued efforts to complete additional GWPP projects annually, have resulted in a need for additional treatment capacity. MSWD's primary treatment plant, the Horton WWTP, is nearing the 80% permitted capacity threshold, warranting the need for additional treatment capacity.[3]

MSWD has elected to pursue the completion of the RWP to meet the growing wastewater treatment capacity needs in its service area. The proposed WVWRF will have an initial capacity of 1.5 mgd to offload existing domestic wastewater treatment plants and accommodate future domestic wastewater flows from the GPP and growth projected over the next 10-years. Ultimately, the WVWRF is planned to have a total capacity of 20 mgd. The WVWRF will initially be designed to produce effluent meeting secondary standards with denitrification for disposal to infiltration basins. The design of the initial

[2] Transport of Contaminates from Wastewater Disposal Systems near Mission Creek Sub Basin, Desert Hot Springs, CA, United States Geological Survey (USGS) and Michigan Technological University, 1996

[3] Per HWWTP Waste Discharge Requirements (Board Order R7-2014-0049), it has a permitted capacity of 2.30 million gallons per day (mgd), with an 80% capacity threshold of 1.84 mgd. Current ow is approximately 1.74 mgd.

facility will have provisions to accommodate a future upgrade to disinfected tertiary effluent standards for water recycle in accordance with California Title 22.

The treatment plant will be located on an undeveloped 60-acre site along Little Morongo Road near 20th Avenue, Desert Hot Springs, California. The land surrounding the plant site is currently undeveloped and zoned light industrial. The site experiences frequent high winds with occasional sand storms. Predominant wind direction is from west to east.

The proposed treatment system must be able to accommodate initial low-flow conditions which are expected to be as low as 0.20 mgd. The plan is to provide aerated biosolids storage and dewatering on the site;

Treatment alternatives were evaluated in the Preliminary Design Report (PDR). The selected treatment will be Activated Sludge treatment using Sequencing Batch Reactors (SBR) for effluent disposal to onsite infiltration basins. The facility will be designed to meet reliability and redundancy criteria established by the State of California based on standards for effluent disposal by on-site infiltration basins and potential future irrigation of public access areas (e.g. golf courses).

The major functional elements of an SBR treatment plant will consist of:

- *Influent Pump Station – submersible pumps, underground wet well*
- *Preliminary Treatment (headworks) – coarse screening and vortex grit removal units (equipment, channels, and tanks covered with off-gas odor control*
- *Anoxic/Aerated Aerobic Tanks with SBR decant tanks, transfer pumps, and waste sludge pumps*
- *Aerated Sludge Storage Tanks and associated blowers and decanters. Tanks and decanters will be covered with off-gas odor control.*
- *Dewatering room with mechanical dewatering in building with off-gas odor control.*
- *Control room, restrooms, locker rooms, and break room.*
- *Laboratory suitable for basic testing in support of daily plant operations.*
- *Chemical Storage (Polymer) – All storage and pumps will be located inside a building.*
- *Depending on the quantities, chemicals will either be delivered by truck and pumped into storage tanks or delivered in totes and off-loaded using a manual or electric pallet jack.*
- *Electrical Room*
- *Emergency Generator (diesel engine).*

Section 8 – Project Delivery Plan

8.1 Procurement Methods

The WVWRF construction is assumed to be competitively bid as a single construction contract. This contracting approach provides a single source of responsibility for the completion of construction, and minimizes the risk of costly and time consuming delay claims. The duration for bidding and award is estimated to be 6 months. This allows 2 months for contractors to submit bids, 1 month to evaluate the bids, and 3 months for contract award.

The pre-purchasing of equipment is not recommended, since the coordination requirements between the equipment manufacturer and contractor are significant with an SBR. Pre-purchasing equipment is a potential source of both schedule delays and monetary claims in this situation.

8.2 Preliminary Construction Schedule

Construction period for this project is estimated to have a duration of 24 months. A summary of the major activities and durations is show in Table 8-1. A more detailed construction schedule will be prepared during detailed design.

Table 8-1 – Project Schedule

Activity	Duration, Calendar Days
Mobilization, submittals, materials delivery	180
Deep structures – excavations and concrete work	150
Other treatment process structures	60
At-grade structures	60
Yard Piping and Duct Banks	60
Install Equipment, electrical cables	60
Site work – Paving, drainage, fencing, lighting, landscaping	45
Building interior fit-out	45
Acceptance Testing	60
Demobilization/Punch list	10
Total	**730**

The schedule includes the sequencing and staging of major work elements highlighted as follows:

- Structures with deep excavations such as the IPS and SBR would start at the beginning of the project following initial mobilization, submittal processing, and materials delivery.
- The coarse screen and grit chamber would be completed after the adjacent deep excavations are backfilled and compacted.
- At-grade structures, such as the admin building, electrical building, chemical storage area, dewatering building, and standby generator, would be started last.
- Major equipment items with long delivery times include the coarse screens, grit removal, SBR system, BFP, odor control system, MCCs, switchgear, and standby generator.
- Major equipment would be delivered and installed before the structures are closed-in.

8.3 Preliminary Opinion of Probable Construction Cost

A summary of the preliminary opinion of probable construction cost (the construction cost estimate) is outlined in this section.

The cost estimate is based on the construction sequencing approach outlined herein, and the allowable time of construction shown on the construction schedule. A detailed cost estimating spreadsheet is attached as Appendix B.

The quantities shown are estimates based on descriptions in this PDR and vendor proposals. At this level of completion, not all work is shown on the drawings. The estimate is intended to be as comprehensive as possible at the preliminary design stage where much of the work is still at a conceptual level.

The quantities for all work items and level of work breakdown structure would be reviewed and updated during the Detailed Design. The cost estimate would be updated with each design submittal and the project level adjustment would be revised with each submittal to reflect increasing levels of completion of the plans and specifications that occur with each submission.

The following factors would be added-on to the estimate to cover the contractor's general office overhead and profit:

- General contractor's field office costs;
- Sales tax;
- General contractor's home office overhead and profit;
- Builders' risk and general liability insurance; and
- Performance and payment bonds.

The allowances for each of these items are outlined in this section.

A summary of the preliminary construction cost for the WVWRF is shown in Table 8-2.

Based on past projects, the following overhead factors are included in the cost estimate:

- General contractor's field office costs are estimated at approximately $29,000 per month over an estimated 24 month construction duration.
- California Sales tax at 7.75% of the estimated materials cost (Desert Hot Springs Rate)
- Contractor's home office overhead and profit at 10% of the total estimated construction cost
- Insurance at 3.0% (1.0% for builder's risk, and 2.0% for general liability insurance)
- Bond cost of 2.0% for the required payment, and performance bonds specified by MSWD on past projects.

The current construction cost estimate is based on June 2018 prices (ENR 20 Cities = 11,000).

A project level allowance of 40% (20% contingency and 20% for project services) is added to the estimated construction cost to establish the total estimated project cost. The project level allowance is broken down as follows

- Contingencies – approximately 20%
- Design Engineering – approximately 6.5%

- Construction Engineering, Contract Administration and Commissioning Support – approximately 10.0%
- MSWD Program and General and Administrative Expenses – approximately 3.5%

The estimated construction cost for the WVWRF is $27.4 million based on current June 2018, Engineering News Record – ENR 20 Cities Index of 11,000). The total estimated project cost is $31.6 million that includes a 40% project level allowance described above.

Table 8-2 – Summary of the Preliminary Construction Cost for the WVWRF

Specification Division	SBR Alternative (no disinfection)
Division 1- Field Office Requirements	696,960
Division 2- Site work	1,723,476
Division 3- Concrete	5,242,963
Division 4- Masonry	503,400
Division 5 -Metals	431,217
Division 6 -Wood and Plastics	131,040
Division 7- Thermal & Moisture Protection	336,960
Division 8- Doors, Windows, & Hardware	36,000
Division 9- Finishes	225,860
Division 10- Specialties	1,275
Division 11- Equipment	4,943,750
Division 12- Furnishings	3,300
Division 13- Special Construction	0
Division 14- Conveying Systems	36,000
Division 15- Mechanical	1,871,006
Division 16- Electrical	1,518,113
Division 17- Instrumentation	549,150
Subtotal-Divisions 1-17	**18,250,470**
Division 0	4,565,414
SUBTOTAL	**22,815,884**
Contingency (20%)	4,563,177
TOTAL ESTIMATED CONSTRUCTION COST	**27,379,061**
DESIGN ENGINEERING (+/- 6.5%)	1,483,032
CONSTRUCTION ENGINEERING (+/- 10%)	2,281,588
PROGRAM AND ADMINISTRATIVE COSTS	456,318
TOTAL 2018 ESTIMATED PROJECT COST	**31,600,000**

Appendix A

Technical Memorandum
GEOTECHNICAL INVESTIGATION

Prepared for

Mission Springs Water District

Mission Springs Water District (MSWD)
West Valley Water Reclamation Facility (WVWRF) Design Project
Desert Hot Springs, CA

AECOM Project: 60551186.1.13
999 Town and Country Road
Orange, CA 92868
T: 714-567-2400 F: 714-567-2594

October 24, 2018

October 24, 2018

Danny Friend, Project Manager
Mission Springs Water District
66575 2nd St,
Desert Hot Springs, CA 92240

Subject: **Geotechnical Investigation**
Technical Memorandum
Mission Springs Water District
West Valley Water Reclamation Facility (WVWRF) Design Project

Dear Mr. Friend,

AECOM Technical Services, Inc. (AECOM) is pleased to provide you with our Geotechnical Investigation Technical Memorandum in support of the proposed West Valley Water Reclamation Facility (WVWRF) Design project.

The scope of work included a site-specific subsurface exploration, laboratory testing, geotechnical engineering analyses, earthwork and foundation recommendations and preparation of this Geotechnical Investigation memorandum. This memorandum presents the findings from our subsurface exploration, our interpretation of the subsurface conditions encountered, the results from laboratory testing, and conclusions and recommendations pertaining to the geotechnical aspects of the design and construction.

We hope this memorandum meets your current project needs. If you require additional information, please contact the undersigned, Praveen Yerra, at (714) 567-2492 or Praveen.yerra@aecom.com

Sincerely,

AECOM

Praveen K. Yerra, P.E.
Project Geotechnical Engineer
PE 81209 Exp. 9/30/19

Christopher W. Goetz, P.G., C.E.G.
Principal Engineering Geologist
CEG 1833 Exp. 07/31/19

Michael G. Smith, P.E., G.E.
Principal Geotechnical Engineer
GE 2229 Exp. 3/31/20

Table of Contents

Appendices

Figures

Tables

ACRONYMS

AECOM	AECOM Technical Services, Inc. (CONSULTANT)
AB	aggregate base
ASTM	ASTM International
bgs	below ground surface
bpf	blows per foot, i.e., blow count
Cal/OSHA	California Occupational Safety and Health Administration
Caltrans	California Department of Transportation
CBC	California Building Code
CGS	California Geological Survey
CMB	crushed miscellaneous base
deg	degree
ft	feet
GLA	GeoLogic Associates
HMA	Hot mix asphalt
ksf	kips per square foot
LCI	Landmark Consultants, Inc.
MSWD	Mission Springs Water District (CLIENT)
OWC	optimum water content
PCC	portland cement concrete
psi	pounds per square inch
pci	pounds per cubic inch
ppm	parts per million
psf	pounds per square feet
PVC	polyvinyl chloride
RCFCWCD	Riverside County Flood Control and Water Conservation District
WVWRF	West Valley Water Reclamation Facility
SPT	Standard Penetration Test
TI	Traffic Index
TM	Technical Memorandum
USEPA	United States Environmental Protection Agency

Section 1 – Introduction

1.1 General

The site comprises two locations in the vicinity of North Palm Springs, California, as shown on Figure 1. The first location, at approximately 33.906721°N, 116.529044°W, is the planned primary location of the proposed Mission Springs Water District (MSWD) West Valley Water Reclamation Facility (WVWRF), southeast of North Palm Springs. The alternate site location, at approximately 33.943012°N, 116.534067°W, is tentatively chosen as an alternative location for off-site spreading basins for discharge of the treated water if the primary location is deemed unsuitable for treated water discharge. The alternate site is located northeast of North Palm Springs.

1.2 Scope and Purpose

This Technical Memorandum (TM) has been prepared to present the results of our geotechnical investigation and recommendations for the design and construction of the proposed WVWRF for MSWD in North Palm Springs, California. This memorandum provides the findings from geotechnical field exploration and laboratory testing, interpretation of the geologic and geotechnical conditions encountered, and recommendations for the proposed WVWRF including the spreading basins.

Our scope-of-work included:

- Review of available geotechnical information.
- Subsurface exploration including infiltration testing and drilling, sampling and logging of hollow-stem auger borings.
- Laboratory testing on selected soil samples.
- Interpretation of the geologic and geotechnical conditions encountered.
- Conducting engineering evaluations and analyses to develop recommendations for the design and construction of the MSWD WVWRF.
- Preparation of this TM that addresses the geotechnical aspects of the proposed WVWRF design and construction.

Appendix A presents the geotechnical boring logs for the current investigation as well as select boring logs that are relevant to this project from previous exploration programs performed by others. **Appendix B** presents the results from our infiltration testing. **Appendix C** presents the laboratory test results. **Appendix D** presents calculations that support the geotechnical recommendations provided in this report.

1.3 Summary of Findings

As part of the scope of work, AECOM collected available geotechnical data and identified locations where additional information was necessary for preliminary evaluation. Based on the identified data gaps, AECOM completed geotechnical borings at 10 locations with depths ranging between 21 feet and 50.5 feet below ground surface (bgs). Temporary wells were installed at four () of the boring locations for the purpose of infiltration testing to study soil permeability characteristics. From the borings, soil samples were collected and tested and site data were analyzed for development of preliminary geotechnical recommendations for the proposed WVWRF.

Summary of findings from this TM:

- Subsurface soils encountered during the field exploration consist of medium dense to very dense silty sands and poorly graded sands with silt and gravel.
- While no cobbles or boulders were encountered during the geotechnical investigation for this project or in the LCI Report (2008), cobble and boulders were encountered throughout MSWD's solar project site, adjacent to (to the north) this site and are indicated on the boring logs from that project (BSK Associates, 2015). Cobbles and boulders were also encountered during the drilling of Well 33 and indicated in the geotechnical report by GeoLogic Associates (GLA, 2004). Therefore, the possibility of encountering cobbles and boulders and difficult excavation conditions is considered likely and contractors bidding the work should consider this in estimating the construction means, methods, schedule and cost.
- Soil is non-corrosive in accordance with the Caltrans corrosion guidelines (Caltrans, 2015).
- Groundwater was not encountered during this geotechnical investigation or in previous investigations. Groundwater level is expected to be as deep as 230 feet bgs. However, the possibility of seasonal fluctuations in groundwater due to precipitation or perched water cannot be discounted.
- Recent publications do not indicate mapped faults crossing the site (CGS, 2010). The potential fault rupture hazard at the site is considered low to moderate. However, the site is likely to be subject to seismic shaking at some time in the future. The subsurface soils at the site correspond to the International Building Code Site Class Type D.
- Shallow foundations are a proposed option for the project site. It is anticipated that all structures will be founded on mat foundations or slabs-on-grade.
- Due to the presence of loose soils at the anticipated bottom of foundation elevation, it is recommended that soils within 3 feet from the bottom of foundation or slab on grade be removed and replaced with structural fill following recommendations in this TM.
- Unsupported temporary slopes with conditions similar to those encountered during the exploration (Cal/OSHA Type C soils) should be made at an inclination no steeper than 1.5:1 (horizontal to vertical), or flatter if field conditions dictate.
- As soil conditions may vary, the contractor should employ an excavation competent person as defined by Cal/OSHA to determine all aspects of excavation safety.
- Majority of the project site soils are suitable for use as structural fill provided it meets the requirements outlined in **Section 5.2.5** of this TM**.**

Section 2 – Geotechnical Exploration and Laboratory Testing

2.1 Field Work

AECOM conducted a geotechnical field exploration at the project site from September 26th through October 4th, 2017. The exploration program advanced a total of ten (10) hollow-stem auger borings with depths ranging between 20 feet and 50 feet bgs. The borings were drilled across both the primary and alternate site locations for the proposed WVWRF. Of these borings, four (4) were developed into temporary monitoring wells with 2-inch diameter slotted PVC pipe for the purpose of infiltration testing. Approximate locations of the borings are presented in **Figures 2a** and **2b,** and a summary of the exploration program is provided in **Table 1**. Boring and monitoring well logs are provided in **Appendix A**.

An AECOM field representative visually classified the soil cuttings and samples in accordance with the Caltrans Soil and Rock Logging, Classification, and Presentation Manual (Caltrans, 2010), and maintained a detailed record of subsurface materials encountered in the exploratory boring. Driven soil sampling was performed at approximately 5-foot vertical intervals to collect soil samples. Due to the granular nature of the subsurface soils, the majority of samples were collected using standard penetration test (SPT) samplers without liners, in accordance with ASTM International (ASTM) D1586 guidelines. When conditions permitted, California samplers (2.42-inch inside diameter) were advanced to collect relatively intact samples. Both SPT and California samplers were driven 18 inches into the subsurface soils using a 140-pound automatic-trip hammer with successive 30-inch drops. The number of blows required to drive the sampler for the last 12 inches was recorded on the boring records.

Temporary monitoring wells were installed in 20-foot deep boreholes with the bottom ten-foot section consisting of 2-inch inside diameter flush-threaded Schedule 40 polyvinyl chloride (PVC) with a 0.010 inch slot size. The top and bottom of the PVC pipe were fitted with flush threaded plugs (cap). The portion above the slotted PVC pipe was fitted with a solid PVC pipe to allow for infiltration testing only in the bottom 11 to 12 feet of the borehole. It is anticipated that the invert of the proposed spreading basins will be approximately 9 to 10 feet below existing ground surface. The approximate screened interval ranged from 9 to 9.5 feet bgs to a maximum depth of 20.9 to 21.5 feet bgs. The annular space around the PVC pipe within the borehole was filled with sand filter pack from the bottom of the borehole to approximately 3 to 4 feet above the screened interval (5 to 6 feet bgs). The filter pack consisted of rounded to sub-rounded graded #2/12 sand. Above the sand pack, 1 foot of ¼-inch bentonite "time release" pellets were placed. The upper 5 feet of annular space above the bentonite pellets zone was grouted using Portland cement/ bentonite slurry. Following construction of the well, water was continuously added to the borehole for approximately 15 minutes to flush any debris out the threaded screens.

Table 1 – Summary of Geotechnical Field Exploration

Boring Number[1]	Maximum Boring Depth (ft)	Latitude (deg.)[2]	Longitude (deg.)[2]	Approximate Ground Surface Elevation (ft)[2]	Temporary Well Installed?
A-17-001	21.5	33.903680°	-116.528750°	717	Y
A-17-002	21.5	33.903920°	-116.530230°	718	Y
A-17-003	41.5	33.907970°	-116.529581°	745	N
A-17-004	32	33.908510°	-116.528919°	750	N
A-17-005	40.3	33.908070°	-116.530740°	746	N
A-17-006	50.1	33.909800°	-116.530211°	761	N
A-17-007	50.2	33.909720°	-116.530800°	760	N
A-17-008	20.9	33.946298°	-116.535102°	1028	Y
A-17-009	50.3	33.942640°	-116.533889°	996	N
A-17-010	21	33.940944°	-116.533340°	984	Y

[1]A – Hollow-stem auger
[2]Locations based on GPS; elevations based on USGS topographic maps and were converted to North American Vertical Datum of 1988.

2.2 Borehole Abandonment

Boreholes that were not developed into temporary wells were backfilled by pumping a mixture of cement and bentonite grout through a tremie pipe that was lowered to the bottom of the borehole. The upper 6 inches near existing ground at each bored hole was capped with soil cuttings to match existing subsurface conditions. The surrounding ground surface was reinstated to match surroundings following borehole completion. The boring logs are provided in **Appendix A.**

2.3 Infiltration Testing

Infiltration tests were conducted at four boring locations: A-17-001 & A-17-002 (primary WVWRF site) and A-17-008 & A-17-010 (alternate off-site spreading grounds site) to evaluate the in-situ soil permeability characteristics. The boring infiltration testing method followed the procedure outlined in "Riverside County – Low Impact Development BMP Design Handbook" for the Santa Ana River watershed by Riverside County Flood Control and Water Conservation District (RCFCWCD, 2011).

Prior to performing the boring infiltration tests, each test hole was pre-soaked for two hours by continuously filling the borehole with clean water. Following the pre-soaking period, the test boring was refilled with water to at least five times the hole's radius. General subsurface conditions at the infiltration boring locations indicate dry loose granular soils with high infiltration rates. This was confirmed in the field as the first two consecutive rates of water drop measurements in the borehole indicate six inches of water dissipates into the surrounding soils through the PVC slots in less than 25 minutes. The drop in water level was measured from the top of casing at pre-determined time intervals. In order to capture a reasonable rate that can be measured in the field, the time interval for water level measurements was adjusted for each of the borehole locations. The time interval between water level readings for each well varied between 4 minutes and 10 minutes due to the quick rate of water level drop in the casings. The hole was refilled with clean water after every reading to the fixed reference point at all test locations (A-17-001, A-17-002, A-17-008 and A-17-010). Occasionally to allow for faster and more accurate measurement of water levels or lack of enough water available for refilling the borehole, measurements were taken without filling the borehole to the top of the casing after every reading. The drop in water level measured is the infiltration rate which relates to the speed at which water progresses downward and laterally through the soil. The test was performed for at least one hour, consisting of at least six measurements taken with a precision of 0.25 inches or better.

Based on the BMP Design Handbook, Appendix A, Section 2.3, the tested infiltration rates are derived converted from the measured percolation rates using the "Porchet Method" (RCFC, 2011). Both the measured percolation rates and the tested infiltration rates are presented in **Appendix B**. The procedure calls for using the last reading as the rate of infiltration. Based on the calculations from various borings, the rate of infiltration at the primary site varied between 5 and 9 inches/hour and the rate varied between 5 and 11 inches/hour at the alternate spreading ground site.

2.4 Laboratory Testing

Laboratory testing was performed primarily at AECOM's geotechnical laboratory in Santa Ana. Select samples were tested to confirm or modify (if necessary) the visual classification of the soils from the field identification, and to evaluate their physical and engineering properties. Tests performed include soil classification (ASTM D2487), water content determination (ASTM D2216), in-situ density (ASTM D7263), Atterberg limits (ASTM D4318), wash analysis (ASTM D1140), sieve analysis (ASTM D6913), direct shear (ASTM D3080), and swell or settlement potential (ASTM D4546).

Corrosivity (Caltrans test methods 417, 422 and 643) tests were performed by the HDR laboratory in Claremont, California, and R-value (Caltrans test method 301) tests were performed by AP Engineering and Testing, Inc. in Pomona, California.

A description of the laboratory testing and the test results are presented in **Appendix C**.

Section 3 – Geology and Subsurface Conditions

3.1 Regional Geology

The project area lies within the Colorado Desert geomorphic province of California. A major feature of the Colorado Desert geomorphic province is the Salton Trough, a seismically active extensional basin influenced by the movement along the San Andreas Fault, which separates the Pacific Plate to the west and the North American Plate to the east. The Salton Trough is a large northwest-southeast oriented basin filled with alluvial sediments that have been shed off the surrounding mountains and subsequently carried down the valley towards the Salton Sea via alluvial fan and fluvial processes (e.g., Mission Creek and Whitewater River drainage systems in Coachella Valley). The Coachella Valley forms the northern part of the basin, which opens up to the much broader Imperial Valley to the southeast. The northeastern side of the basin is bound by the Little San Bernardino Mountains and the Chocolate Mountains. The southwestern side of the basin is bound by the San Jacinto Mountains and the Santa Rosa Mountains. The surrounding mountains are typically composed of crystalline basement rock. The material filling the basin is predominantly Quaternary aged alluvial fan, fluvial and lacustrine deposits. Early Quaternary/ late Tertiary sedimentary deposits crop out forming small hills within the valley as geomorphic expressions of the San Andreas Fault (CDMG, 1965). **Figure 3** shows a regional geologic map of the project site. Local fault strands from the San Andreas Fault system are also shown.

The proposed locations of the WVWRF and off-site spreading basins lie within the northwestern end of the Coachella Valley. The site is on a gentle south-sloping alluvial fan surface within the general influence of the Mission Creek Drainage. A primary wash of the Mission Creek drainage system lies approximately 0.15 miles to the east of the WVWRF site. The subsurface deposits at the site are derived from Late Holocene alluvial wash deposits (Qw) and Holocene to Late Pleistocene alluvial valley deposits (Qya) (California Geological Survey [CGS], 2012).

3.2 Project Site Soils

Subsurface conditions were examined based on the recent AECOM subsurface investigation and a review of boring logs from previous investigations performed at MSWD Well 33 (GLA, 2004; LCI, 2008). **Figures 4a and 4b** show the proposed main WVWRF and the alternative off-site spreading basins are underlain by alluvial soils. The alluvial soils are typically medium dense to very dense silty sands and poorly graded sands with silt and gravel.

At the primary WVWRF location, two layers of alluvium can be distinguished based on the subsurface material properties. The upper alluvium layer is composed predominantly of medium dense to dense poorly graded sand with silt and loose to dense well-graded sand with silt. A thin layer of medium dense sandy silt was observed at boring A-17-003. The lower alluvium layer is denser, has slightly lower water content, and increased content of fines. The lower alluvium layer is composed predominantly of medium dense to very dense silty sand, poorly graded sand with silt, and well-graded sand with silt. The uppermost 3 feet of alluvial soils are found to be very loose, and will require removal during excavation. Details on other excavation considerations are located in **Section 5.2.2**.

At the off-site location, highly variable alluvium consisting of medium dense to very dense poorly graded sand with silt and silty sand, and very dense well-graded sand with silt are found to the maximum depths explored (20.9 to 50.5 feet bgs). Generalized subsurface profiles at the proposed primary WVWRF site and the alternative off-site location for the spreading basins are presented in **Table 2** and **Table 3**, respectively.

Table 2 – Generalized Subsurface Material Properties – Primary MSWD WVWRF Site

Geologic Unit	Soil Description	Approximate Depth bgs	SPT N_{60}[2,3] Values (bpf)	Index Properties: Water[2] Content (%)	Index Properties: Dry Unit Weight[2] (pcf)	Index Properties: Fines Content[2] (%)
UPPER ALLUVIUM	Med. dense to dense Poorly-graded Sand with Silt (SP-SM); Loose to dense Well-graded Sand with Silt (SW-SM)	Northern extent: 0-15ft; Center: 0-20ft Southern Extent: 0-14ft.	Granular Soil			
			12 to 51 (28)	<1 to 22 (2)	110-118 (115)	5 to 9 (7.5)
	Med. dense Sandy Silt (ML)	Center: 20-22ft	Fine Grained Soil			
			26 (26)	--	--	--
LOWER ALLUVIUM	Med. dense to v. dense Silty Sand (SM), Med. dense to v. dense Poorly-graded Sand with Silt (SP-SM), Med. dense to v. dense Well-graded Sand with Silt (SW-SM); Dense to v. dense Poorly-graded Sand (SP)	Northern extent: Elev. 15-50ft; Center: 15-40ft Southern Extent: 15-20ft	Granular Soil			
			25 to 100 (55)	<1 to 10 (1)	--	4 to 49 (13)
	V. dense Sand with Silt (SP-SM)	Northern extent: 45-50ft	Clayey Soil			
			65 (65)	<1 (<1)	--	--

Notes:
(1) Subsurface profile based on borings A-17-B1 through A-17-B7.
(2) Test values shown in low-high range with average value in parenthesis.
(3) SPT-N_{60}: SPT blow count adjusted for standard hammer efficiency of 60%.
(4) bpf: blow counts per foot; pcf: pounds per cubic foot; psf: pounds per square foot; ksf: kips per square foot.

Table 3 – Generalized Subsurface Material Properties – Alternative Off-site Spreading Basins

Geologic Unit	Soil Description	Approximate Depth bgs	SPT N_{60}[2,3] Values (bpf)	Index Properties: Water[2] Content (%)	Index Properties: Dry Unit Weight[2] (pcf)	Index Properties: Fines Content[2] (%)
			Granular Soil			
ALLUVIUM	Med. dense to v. dense Poorly graded Sand with Silt (SP-SM), V. dense Well-graded Sand with Silt (SW-SM), Med. dense to v. dense Silty Sand (SM)	0-50ft	17 to 100 (76)	0 to 2 (<1)	98 to 112 (105)	6 to 21 (10)

Notes:
(1) Subsurface profile based on borings A-17-B8 through A-17-B10.
(2) Test values shown in low-high range with average value in parenthesis.
(3) SPT-N_{60}: SPT blow count adjusted for standard hammer efficiency of 60%.
(4) bpf: blows per foot, i.e., blow count; pcf: pounds per cubic foot; psf: pounds per square foot; ksf: kips per square foot.

Two prior subsurface investigations were performed at the project site. The first report was completed by GeoLogic Associates in September, 2004 and is titled "Geotechnical Report, Garnet Well Suction Reservoir, Mission Springs Water District, Desert Hot Springs, Riverside County, California." One soil boring was performed to 30.5 feet bgs during this investigation. Well-graded sand with gravel with increasing gravel content starting at 19 feet bgs was reported in the boring. Blow counts indicated medium dense materials above approximately 15 feet bgs, and dense materials below. No groundwater was encountered during drilling.

The second subsurface investigation was performed by Landmark Consultants, Inc. in April 2008 titled "Geotechnical Investigation Report, Proposed Future Regional Wastewater Facility, Desert Hot Springs, California." The investigation included ten soil borings across the site that varied in depth from 38.5 to 51.5 feet bgs. Materials reported on the borings logs were a combination of poorly graded sand, silty sand, gravelly sand and gravelly silty sand. A thin interbed of sandy silt was reported in boring B-2. Apparent densities of the material ranged from medium dense to dense with few very dense layers. The very dense layers typically occurred in gravelly deposits, and the high blow counts are likely more a reflection of the gravel content than of the soil's relative density. No groundwater was encountered in any of the borings during drilling to the maximum depth explored of 51.5 feet bgs.

Cobble and boulders were encountered throughout MSWD's solar project site, adjacent to (to the north) this site and was indicated on the geotechnical report prepared for that project (BSK Associates, 2015). Majority of the borings for the solar project encountered refusal due to cobbles between 5 feet and 17 feet bgs. Significant amounts of cobbles and boulders were encountered during the installation of short c-channel piles at MSWD's solar project site, adjacent (to the north) to this site. The piles were driven to a maximum depth of 8.5 feet bgs, and cobbles and/or boulders were encountered at approximately 102 of the 620 locations. Based on information available from MSWD representatives who provided construction observation for the construction of the pile foundations, the cobbles and boulders prevented pile driving and had to be removed by excavation. Cobbles and boulders are also evident from the drilling log for well No. 33, where cobbles and boulders were encountered continuously from the ground surface up to a maximum depth of 150 feet bgs. The geotechnical report by Geologic Associates (GLA, 2004) prepared for Well No. 33 indicated cobbles to the maximum depth explored of 30 feet bgs.

3.3 Groundwater

Due to multiple splays of the San Andreas Fault transecting the Coachella Valley, the alluvial groundwater aquifer is split into multiple sub-basins (MWH, 2013). The project site lies within the Garnet Hills Sub-basin. Groundwater data from 2009 suggest groundwater elevations in the project vicinity are between 500 and 600 feet. These elevations correspond to a depth to water between 130 and 230 feet below ground surface. Groundwater level measurements from the production well on the north end of the project site shows levels deeper than 150 feet bgs.

Groundwater was not encountered during the previous field investigations in 2004 (GLA, 2004) and 2008 (LCI, 2008) to the maximum depth explored of about 30.5 feet and 51.5 feet below ground surface (bgs), respectively, corresponding to approximately 725.5 feet and 688.5 feet in elevation (National Geodetic Vertical Datum of 1929). Groundwater was not encountered during the recent borings performed for the subject investigation, to the maximum depth explored of about 50.3 feet bgs, at approximately 709 feet elevation (National Geodetic Vertical Datum of 1929). However, the possibility of seasonal fluctuations in groundwater due to precipitation or perched water cannot be discounted.

3.4 Corrosivity

Corrosivity testing was completed as part of this investigation to assess the corrosion potential of the soils. The corrosion tests were completed in accordance with Caltrans test methods and United States Environmental Protection Agency (USEPA) test methods. The results are summarized in **Table 4.**

Table 4 - Summary of Corrosivity Test Results

Boring	Depth (feet)	Approximate Elevation (feet NAVD88)	pH Threshold ≤ 5.5	Minimum Resistivity (Ohm-cm) Threshold ≤ 1,000	Sulfate Content (ppm) Threshold ≥ 2,000	Chloride Content (ppm) Threshold ≥ 500
A-17-003	10	760	9.8	9,600	17	2.0
A-17-006	10	761	11.4	2,400	139	4.5

Notes:
(1) ppm = parts per million. ND = Non Detectable. ohm-cm = ohm-centimeter.
(2) Resistivity is not a corrosion criterion, but an indicator of soluble salts per Caltrans Corrosion Guidelines (Caltrans, 2015).

Caltrans (Caltrans, 2015) considers a site to be corrosive to foundation elements if one or more of the following conditions exist for the soil samples taken from the site:

- Chloride concentration is greater than or equal to 500 parts per million (ppm),
- Sulfate concentration is greater than or equal to 2,000 ppm,
- PH is 5.5 or less.

The minimum resistivity can be an indicator for the relative quantity of soluble salts present in the soil or water. In general, a minimum resistivity value less than 1,000 ohm-cm indicates high soluble salts and higher propensity for corrosion. However, since sulfate and chloride contents were measured, the minimum resistivity is considered an indicator only.

Based on the results of the corrosivity testing, the site is interpreted to be non-corrosive in accordance with the Caltrans corrosion guidelines (Caltrans, 2015).

Section 4 – Geotechnical Evaluations

4.1 Seismicity and Faulting

A summary of the preliminary geotechnical findings is presented below. The WVWRF site lies approximately 400 feet southwest of an Alquist-Priolo Fault Zone pertaining to the San Bernardino Mountain Section of the South Branch of the San Andreas Fault (CDMG, 1980). The fault is considered active within the Holocene time period (CGS, 2010). The mapped fault trace itself lies as near as approximately 1,000 feet northeast of the site of the proposed WVWRF. The North Branch of the San Andreas Fault lies 3.5 miles to the northeast of the site and is also an Alquist-Priolo Zoned Fault. The Garnet Hill Fault is considered a potentially active strand of the San Andreas Fault and lies approximately 0.65 miles to the southwest of the site. Recent publications do not indicate mapped faults crossing the site (CGS, 2010). The potential for fault rupture at the site is considered low to moderate.

The WVWRF site location with respect to nearby faults, as generated by Caltrans' ARS Online Tool (Caltrans, 2017), is shown on **Figure 5**. The South Branch of the San Andreas Fault (referred to as San Bernardino South) is the closest fault and could generate the highest ground motion. The San Andreas San Bernardino South is a strike-slip fault with a vertical (90 degree) orientation. Caltrans has assigned the fault a maximum earthquake magnitude (M_{max}) of 7.9. A summary of fault parameters and distances to this fault and two others for both the primary WVWRF site and the alternative off-site spreading basins site are presented in **Table 5**.

Table 5 – Seismic Parameters for the Significant Faults in the Site Vicinity

Faults (Caltrans Fault ID)	Maximum Earthquake Moment Magnitude (M_{max}) [1]	Fault-Site Distances to WVWRF[1]	Fault-Site Distances to Alternate spreading basins[1]	Fault Type[1]
San Andreas[2] (San Bernardino South Segment) (325)	7.9	0.80 km (0.5 miles)	2.80 km (1.75 miles)	Strike-Slip
San Andreas[2] (San Gorgonio – Garnet Hill Segment) (358)	6.7	1.5 km (0.95 miles)	4.6 km (2.85 miles)	Strike-Slip
San Andreas[2] (San Bernardino North Segment) (294)	7.4	6.0 km (3.7 miles)	3.5 km (2.2 miles)	Strike-Slip

[1]Obtained from Caltrans ARS Online, v2.3.09 (2017)
[2]This fault is a blind thrust fault that does not rupture the ground surface. The distance noted is the distance to the upper limit of the rupture plane in the subsurface provided by Caltrans ARS Online.

4.2 Seismic Parameters

The site will likely be subject to seismic shaking at some time in the future. Seismic ground motion parameters were developed using the USGS website, U.S. Seismic Design Maps. The site coordinates used in the analysis were 33.90605° north latitude, -116.52902° west longitude, which pertains to the primary WVWRF site where new structures are planned. The subsurface soil at the WVWRF site corresponds to International Building Code Site Class Type D based on the average Vs30 of 270 meters per second obtained for the site (Vs30 is the time-averaged shear-wave velocity (Vs) in the upper 30 meters).

Table 6 - Seismic Design Parameters

Parameter	Factor	Value
Mapped Spectral Response Acceleration (0.2 sec Period)	S_S	3.029g
Mapped Spectral Response Acceleration (1.0 sec Period)	S_1	1.222g
Site Class	Site Class	D
Site Coefficient	F_a	1.0
Site Coefficient	F_v	1.5
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec Period)	S_{MS}	3.029g
Maximum Considered Earthquake Spectral Response Acceleration (1.0 sec Period)	S_{M1}	1.833g
Design Spectral Response Acceleration (0.2 sec Period)	S_{DS}	2.020g
Design Spectral Response Acceleration (1.0 sec Period)	S_{D1}	1.222g
Seismic Design Category: D		

It should be recognized that much of southern California is subject to some level of damaging ground shaking as a result of rupture along the major active (and potentially active) fault zones that characterize this region. Design utilizing the 2016 California Building Code (CBC, 2016) is not meant to completely protect against damage or loss of function. Therefore, the preceding parameters should be considered as minimum design values.

The alternate site will only have discharge basins constructed very close to the surface and they are not considered to be structures that will require seismic design.

4.3 Slope Stability

The topography at the site is relatively flat with very gentle slopes. Due to the relatively flat-lying topographic character of the site, the potential for slope failure is considered low.

For the planned discharge basins, slopes inclined 2:1 (horizontal: vertical) or flatter are considered grossly stable. At this time, detailed design of the planned basins is not available for slope stability analysis. Once the design is available, specific slope stability analysis can be performed and recommendations can be refined.

4.4 Liquefaction

Liquefaction is a phenomenon in which loose to medium dense, saturated, granular materials undergo matrix rearrangement, develop high pore water pressure, and lose significant shear strength because of cyclic ground vibrations induced by earthquakes. This rearrangement and strength loss is followed by a reduction in bulk volume of the liquefied soils. The effects of liquefaction can include development of sand boils, the loss of bearing capacity below foundations, settlement in level ground, large horizontal deformations of relatively level ground with an unconfined vertical face (referred to as lateral spreading) and instability in areas of sloping ground (also known as flow sliding). Liquefaction is generally considered to occur only within about 50 feet of the ground surface.

Due to the lack of presence of groundwater in the upper 50 feet of subsurface, the potential for liquefaction at the site is considered low.

4.5 Tsunami

Tsunamis are large waves in the ocean typically caused by submarine earth processes such as earthquakes, coastal landslides or volcanic eruptions. Tsunamis can travel thousands of miles across the ocean and present a serious hazard to coastal developments. The degree of this hazard strongly depends on the size and type of the source of the tsunami, the exposure of the project site to the open ocean and the direction from which the tsunami is coming. The site has no coastal exposure and therefore very low potential for tsunami hazard.

4.6 Expansion Potential

The on-site material predominantly consists of granular soils. Expansive soils are typically fine grained. Potential for expansive soils should be considered low.

4.7 Collapse Potential

At their dry, natural state, soils with collapse potential possess stiffness and high apparent shear strength; but upon wetting, they could exhibit a significant decrease in volume (described as collapse, hydro-consolidation, or hydro-compression). Such soils, which exhibit this phenomenon at fairly low stresses, are called collapsible soils. Collapsible soils are generally characterized by their loose structure of bulky shaped grains, often in the silt to fine sand size with a small amount of clay. There may be only slight cementing agents such as calcium carbonate, salts and dried clay, with combinations being common. Geologic materials with collapse potential consist of Aeolian, fine alluvial fan deposits, mud flows, flash flood deposits, loosely place fills, and some types of residual soils. Collapse potential is evaluated in terms of collapse index in the laboratory using ASTM D4546, wherein a soil sample is seated in a consolidation apparatus and loaded dry to a selected pressure, then saturated. The collapse potential is defined as the ratio of change in height of a specimen to the original height of the specimen determined at any stress level after wetting of a soil sample and duplicating the in-situ soil conditions of overburden stress and pore water pressure. Collapse Index is very similar to Collapse potential except it is measured at a vertical stress of 2 tons per square feet (tsf) and is used to describe degree of collapse under specified conditions. Table 7 summarizes Collapse Potential Test (ASTM D4546) results for this project:

Table 7 – Summary of Collapse Potential Test Results

Boring ID	Site Location	Sample Depth (ft)	USCS Soil Type	Final Water Content (%)	Final Dry Unit Weight (pcf)	Collapse Potential (%)
ACM-17-B4	WVWRF	5	SW-SM	14	112	0.3
LCI-08-B1	WVWRF	20	SM/SP	17.5	113	0.1
LCI-08-B2	WVWRF	30	SM/SP	21	106	0.2
LCI-08-B7	WVWRF	22	SM/SP	21	107	0.6

Notes:
LCI – Landmark Consultants, Inc. (2008), ACM – AECOM Technical Services (2017)

Table 8 – Degree of Collapse and Ranges of Collapse Index

Degree of Collapse	Collapse Index (%)
None	0
Slight	0.1 to 2.0
Moderate	2.1 to 6.0
Moderately Severe	6.1 to 10.0
Severe	>10

[1]Collapse classification index in accordance with ASTM D5333-03

Based on laboratory test results from current and previous investigations, the site has slight collapse potential.

4.8 Scour

Scour was not considered a design issue at this site. The foundations are not located in rivers/creeks or drainage channels.

Section 5 – Conclusions and Recommendations

5.1 General

Based on the results of our geotechnical investigation and our understanding of the project requirements, the site can be developed for its intended purpose provided the recommendations in this report are incorporated in the design and implemented during earthwork and construction of the project.

Recommendations for earthwork, foundation design, seismic design, floor slab support, pavement design, and corrosion protection considerations are presented below.

5.2 Earthwork

Earthwork should be performed in accordance with the applicable portion of the grading code of the State of California, the City of Desert Hot Springs as well as the recommendations of this report, under the observation and testing of the Geotechnical Engineer. Temporary cut and fill slopes should not be steeper than 1.5:1 (horizontal to vertical).

5.2.1 Site Clearing, Grubbing and Stripping

Prior to starting earthwork, the areas to be excavated, to receive fill, or to receive stockpile materials should be cleared, grubbed and stripped of all topsoil, organic material, vegetation, rubbish, deleterious material, and debris resulting from site demolition (if any). Cleared and grubbed material, as well as all rubble waste that may be encountered or created, should be disposed of offsite. All active or inactive utilities within the construction limits should be identified, marked and relocated, while abandoned utility lines should be removed or backfilled.

The project geotechnical consultant should be notified at the appropriate times to provide observation and testing services during clearing, grubbing and stripping operations to verify compliance with the above recommendations. In addition, should any buried structures or unusual or adverse soil conditions be encountered during grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of the project geotechnical engineer for corrective recommendations.

5.2.2 Temporary Excavations

Excavations during construction should be performed in accordance with applicable local, state, and federal regulations including the current California Occupational Safety and Health Administration (Cal/OSHA) excavation and trench safety standards. Unsupported temporary slopes with conditions similar to those encountered during the exploration (Cal/OSHA Type C soils) should be made at an inclination no steeper than 1.5:1 (horizontal to vertical), or flatter if field conditions so dictate. Surcharge loads from vehicle and equipment parking and traffic, excavated materials, stockpile materials or other sources should be set back from the top of the temporary excavation a horizontal distance equal to or greater than 1.5 times the depth of the adjacent excavation.

Trench excavations might be required for utility lines. Based on available data, the upper few feet of soil are predominately loose, dry and cohesion less soils of low fines content. Temporary excavation sidewalls and utility trench walls, even if less than 4 feet high, might pose a life-threatening cave-in danger if excavated with vertical walls. The contractor's excavation competent person, as defined by Cal/OSHA, should determine all aspects of any trench excavation safety.

Based on our exploratory borings for this investigation, no groundwater was encountered. Therefore, we do not anticipate the need for construction dewatering. However, the possibility of seasonal fluctuations in groundwater due to precipitation cannot be discounted. If groundwater is encountered, dewatering will be required. Surface drainage should be controlled along the top of temporary excavations to prevent wetting of the soils and erosion of the excavated faces. Even with the implementation of these recommendations, sloughing of the walls and slopes of temporary excavations may still occur, and workers should be adequately protected.

It is anticipated that the on-site soils can provide suitable support for underground utilities and piping that may be installed for this project. Any soft, loose and/or unstable material encountered at the bottom of excavations for such facilities should be removed and replaced with an adequate bedding material. A non-expansive granular material with a sand equivalent greater than 20 should be used for bedding and shading of utilities.

Significant amounts of cobbles and boulders were encountered during the construction of MSWD's solar project site, adjacent (to the north) to this site. It is also evident from the well No. 33 drilling log where cobbles and boulders were encountered. Refer to Section 3.2 for further details.

Based on information available from MSWD representatives who observed the construction of pile foundations to support the solar panels for the MSWD solar project site, it was noted that several of the piles encountered refusal during pile driving and warranted excavation to remove large rocks and boulders.

We anticipate the construction excavation slopes to be temporarily stable, provided the above recommendations are followed. However, modifications to these recommendations may be required based on observations of the actual conditions exposed in the field or the findings of the contractor's competent person. Our temporary excavation recommendations are provided only as general guidelines; as soil conditions may vary, the contractor should employ an excavation competent person as defined by Cal/OSHA to determine all aspects of excavation safety. The design and construction of temporary excavation support systems (e.g., shoring) and temporary slopes, as well as the maintenance and monitoring of these works during construction, are the responsibility of the contractor. All work associated with temporary excavations should meet the minimal requirements as set forth by the California Occupational Safety and Health Administration (Cal/OSHA). Unsupported temporary slopes with conditions similar to those encountered during the exploration should be made at an inclination no steeper than 1.5:1 (horizontal to vertical), or flatter, as field conditions dictate. Trench excavations should be made with nearly vertical sides, using sheeting and shoring whenever required. All excavation should be observed by a geotechnical engineer of record or a representative so that any necessary modifications based on variations in soil conditions encountered can be performed in an efficient manner. Soils encountered during our field investigation are expected to be excavatable using conventional excavation and grading equipment. All applicable safety requirements and regulations, including Cal/OSHA regulations, should be satisfied. Locally, there is a potential for cobbles, boulders, or cemented materials that may require hard excavation. Raveling of gravel and cobbles should be expected in excavations and could pose a potential safety concern to the construction personnel.

For design purposes, an equivalent fluid weight of 37 pcf, based on an active lateral earth pressure condition, may be used to estimate lateral earth pressure above the groundwater. For portions subject to submergence below groundwater (if encountered), use 17 pcf of equivalent fluid pressure along with the hydrostatic pressure. Hydrostatic pressure should be added to the active earth pressure where the shoring will be submerged.

Surcharge pressures (dead or live) should be added to the above lateral earth pressures where surcharge loads may be located adjacent to the shoring. Surcharge pressures should be applied as a uniform (rectangular) pressure distribution by using a lateral earth pressure coefficient of 0.35. The above coefficient assumes a uniform surcharge load.

Surcharge loads from vehicle/equipment parking and traffic or stockpile materials should be set back from the top of the temporary excavation a horizontal distance equal to at least 1.5 times the depth of excavation. Surface drainage should be controlled along the top of temporary excavations to prevent wetting of the soils and erosion of the excavated faces. Even with the implementation of these recommendations, sloughing of the surface of temporary excavations may still occur, and workers should be adequately protected. In any event, excavation and personnel safety during construction is the sole responsibility of the Contractor.

Care should be taken during shoring removal to prevent creation of voids on the face of excavations. If large voids are created during removal, they should be filled with cement slurry or other approved grout mix.

5.2.3 Over excavation

Due to the presence of loose granular soils with high percentage of silt and clay material at the anticipated bottom of footing elevation, it is recommended that soils within 3 feet from the bottom of foundation or slab on grade and soils within 3 feet of the original ground surface be removed and replaced with structural backfill following recommendations in this TM. The compacted fill should extend a minimum of 5 feet beyond the edges of foundation. The proposed structure may be supported on mat foundations bearing on compacted Structural Fill. It is recommended that "Structural Fill" be used within structural zones[1] beneath all foundations and floor slabs.

Excavations during construction should be performed in accordance with applicable local, state, and federal regulations such that excessive ground movement and failure will not occur. Where space permits and provided that adjacent structures, utility lines, etc. are adequately supported, open excavations may be considered for construction of the project.

1 A structural zone is defined as the space located below a structure or beneath the planes that pass through the bottom of the structure's perimeter footings / exterior walls and that are inclined at 1 horizontal to 1 vertical (increasing the horizontal distance from the structure with increasing depth).

5.2.4 Subgrade Preparation

After performing planned excavation and any required over-excavation and prior to placing any Structural Fill, the ground surface within the building footprint should be observed by the geotechnical engineer to confirm that satisfactory subgrade soils have been encountered. If unsatisfactory soil is encountered at the bottom of excavation or natural ground surface, additional removals may be required. The bottom of the exposed excavation should be scarified to a depth of at least 6 inches, moisture conditioned (as necessary) to above the optimum water content (OWC), and then compacted in-place to at least 95% relative compaction as determined by ASTM D1557 at 0 to 3 percentage points above OWC prior to placing compacted fills. Relative compaction is a measure of the degree of soil compaction and is defined as the ratio of the in situ dry density (or unit weight) divided by of the material's maximum dry density (or unit weight) measured by a reference test procedure (ASTM D1557 for this project). Following the scarification, moisture conditioning and compaction process, the subgrade should be proof rolled, probed and tested as appropriate. Proof rolling should involve making several passes over a subgrade with heavy roller equipment.

5.2.5 Structural Fill beneath Structures

The site soils excavated from the project site are generally considered suitable for use as Structural Fill provided they do not contain rocks or hard lumps greater than 3 inches in maximum dimension and have at least 80 percent passing the ¾-inch sieve, at least 25 percent passing the No. 4 sieve and less than 10 percent passing the No. 200 sieve. It is recommended that "Structural Fill" be used beneath all foundations and floor slabs. Structural Fill materials shall be free of organic material, debris, or other deleterious materials. Materials greater than 1 inch in size shall be placed such that they are completely surrounded by compacted finer soils. Backfill material such as pea gravel and crushed rock do not meet the requirements for structural fill due to their relatively high permeability and potential to store water.

Structural Fill materials should have a minimum sand equivalent of 20 and an Expansion Index of 20 or less when tested in the laboratory in accordance with ASTM D2419 and D4829, respectively. Based on the results of the field exploration and laboratory testing it is concluded that the some of the onsite soils satisfy the requirements of structural fill.

It is noted that backfill material such as pea gravel and crushed rock do not meet the requirements for Structural Fill. This is because the clean rock materials have relatively high permeability and thereby provide the potential to store water. Permeable material should be reserved for below-grade walls or structures that have an appropriate means of drainage discharge at the base of the zone of permeable material.

5.2.6 Fill Placement and Compaction

The maximum dry unit weight of the fill materials should be measured in accordance with ASTM D1557. The field unit weight of fill should be measured in accordance with the sand cone method (ASTM D1556) or the nuclear method (ASTM D6938). The fill materials should be placed in lifts not exceeding 8 inches in depth. The Structural Fill should be compacted to 95% relative compaction as determined by ASTM D1557 at 1 to 3 percent over OWC.

Structural Fill material should be placed in lifts no greater than 8 inches, loose measurement. The water content of the fill material at the time of compaction should achieve uniform moisture between 1 and 3 percent above its OWC. Particles larger than 1 inch for Structural Fill should be placed so that they are completely surrounded by compacted finer soils.

5.2.7 Trench Wall Stability

Trench wall instability will be dependent on the soil and rock properties in the areas of excavations. Shallow groundwater typically contributes to collapse of fill or alluvial soils due to wetting. Extremely dry cohesionless soil, which lacks the apparent cohesion provided by capillary suction, may run on slopes or collapse with even low excavation faces. Wedge failures can occur in the trench walls under such conditions. Shoring is anticipated to be required where trenches cross existing pavements and/or where adjacent utilities exist that cannot withstand lateral movements of the trench walls.

Trench excavations that are made with nearly vertical sides can typically remain open for minutes to hours until positive sidewall shoring/support can be installed. However, this may not be true in areas that transmit groundwater, where existing loose trench backfills exist, where relatively clean, coarse-grained soils are present (such as poorly-graded sand, well-graded sand, poorly-graded gravel and well-graded gravel soil types). In all cases, the contractor should select an excavation, dewatering, and/or shoring scheme that will protect adjacent improvements, including buried utilities.

5.2.8 Trench Preparation and Backfill

5.2.8.1 General Considerations

We anticipate that shallow trenching can be done by conventional trenching machines or power shovels. This opinion is based solely on our knowledge of general geotechnical conditions and on observations made in the exploratory borings.

Minimum trench dimensions are usually specified to allow proper placement of the pipe and backfill. The trench bottom width should be at least 12 inches greater than the pipe outside diameter; unless the contractor can demonstrate that he is able to otherwise place the pipe and backfill to the Owner's satisfaction.

5.2.8.2 Subgrade Preparation

The bottom surfaces of all excavations to receive bedding/fill should be scarified to a depth of at least 6 inches, moisture conditioned, if necessary, and compacted to at least 90 percent relative compaction (as per ASTM Standard D1557) at 1 to 3 percentage points over OWC prior to placing compacted bedding/fills. Following the scarification process, the subgrade should be observed, probed and tested as appropriate. All identified loose or soft zones should be compacted in-place or excavated and replaced with properly compacted backfill to the satisfaction of the geotechnical engineer-of-record in order to establish a competent subgrade on which to place compacted bedding/fill.

5.2.8.3 Pipe Bedding

Bedding is defined as the supporting material placed below the pipe and should have a minimum thickness of 6 inches. To provide uniform and firm support for the pipe, compacted granular materials, such as clean sand, gravel or ¾-inch crushed aggregate or crushed rock, may be used as pipe bedding material. The type and thickness of the bedding material should be chosen based on the proposed type of pipeline to be installed.

The bedding material above the pipe should consist of sand or other granular material conforming to the requirements of Section 306-1.2.1 of the Greenbook.

5.2.8.4 Compaction of Bedding

The maximum dry unit weight of the bedding material should be measured in accordance with ASTM D1557. The field unit weight of bedding should be measured in accordance with the sand cone method (ASTM D1556) or the nuclear method (ASTM D6938). In a narrow trench, use of conventional compaction equipment may be challenging. Verification of appropriate compaction of the bedding material below the spring line is generally difficult by testing. So care should be taken that appropriate densification of the material is performed by visual observation of the moisture conditioning and compaction operations.

5.2.8.5 Pipe Zone and Final Backfill

The pipe zone is the part of the trench from the bedding to a horizontal level 12 inches above the top of the pipe for the full width of the trench. Materials for pipe zone backfill should consist of imported material or on-site material that meets the following requirements. The material should not contain rocks or hard lumps greater than 1 inch in maximum dimension; at least 80 percent (by weight) of its particles should pass through a ¾-inch sieve; and it should have less than 5 percent passing the No. 200 sieve. Final backfill material with a sand equivalent value of 20 or greater and expansion index less than 20 is recommended. The material used for backfill within the pipe zone should be uniformly graded to avoid migration of soil fines into voids and clogging. Perishable, spongy, hazardous, or other undesirable materials should not be used as fill. Clean sands should be placed to surround the pipe completely and minimize voids. Mechanical compaction equipment may be used where feasible.

Materials for the final backfill zone should consist of imported material or on-site material that meets the following requirements. Final pipe zone material does not contain rocks or hard lumps greater than 6 inches in maximum dimension; has at least 80 percent (by weight) of its particles passing through a ¾-inch sieve; and has less than 30 percent passing the No. 200 sieve. Materials greater than 1 inch in size should be placed so that they are completely surrounded by compacted finer soils. Nesting of rocks will not be permitted. To avoid migration of soil fines from the final backfill zone material to the pipe zone material, filter fabric may be placed at the interface at the discretion of the designer or the owner.

5.2.8.6 Imported Materials

Imported soils may be used for pipe bedding and pipe zone backfill. The imported soil should be uniformly graded and should not contain rocks or hard lumps greater than 6 inches in maximum dimension (3 inches if within the upper 18 inches below planned roadway) if placed in the final backfill zone, or a maximum of 1 inch if placed in the pipe zone or bedding zone. It is recommended that the material have a sand equivalent of 20 or more; a low potential for expansion (expansion index less than 20); and less than 30 percent passing the No. 200 sieve. The materials should be free of organic material, debris, man-made materials, or other deleterious materials.

5.3 Infiltration Basin

5.3.1 Design Recommendations

Based on the results of AECOM's infiltration testing, an average infiltration rate of 5 to 9 inches/hour can be used for sizing of the infiltration basin. Based on this infiltration rate and anticipated capacity demand, calculations should be performed to establish the size (footprint dimension) of the proposed basins. It is suggested that it be assumed that infiltration does not occur through fill areas; i.e., the embankments and the adjacent area of ground preparation discussed in Section 5.3.3. Based on preliminary design, the proposed basins are anticipated to consist of a shallow excavation surrounded by fill embankments up to a maximum height of 5 feet above the existing grade. Anticipated water level within the basin is anticipated to be approximately 1 foot above the bottom of the basin.

It should be anticipated that initially, the rate of infiltration will be somewhat closer to the design rate since the majority of flow will be in the vertical direction. The infiltration rate may reduce as the water encounters fine-grained layers and water is forced to move laterally away from the footprint of the basin. With time the infiltration rate may decease due to sedimentation and other deposits; periodic cleaning and furrowing may help restore infiltration to near initial rates. The infiltration rate and performance of the infiltration basin greatly depends on various other factors such as the frequency at which the water will be discharged into the basin, rate of inflow into the basin, duration of each discharge, and degree of maintenance of the basin bottom. It is anticipated that the water discharge into the basin is generally clean and treated water free of debris.

Further design considerations and recommendations are presented below:

- The bottom of the infiltration basin should be installed entirely in undisturbed natural ground. Therefore, the areas recommended for construction of the subsurface disposal systems should remain in an undisturbed, natural condition.

- Excessive travel over the footprint area at the bottom of the proposed excavation with heavy grading and construction equipment should also be avoided. It is also recommended that the construction of the basin embankments be performed using smaller and lighter equipment such as excavators. Heavier excavation equipment such as dozers, front end loaders or scrapers should be placed in unexcavated areas.

- The subsurface infiltration basin-disposal systems should not be located within 15 feet of any 100-year flood limits or within 15 feet of any principal drainage.

- It is imperative that the infiltration basin pits be observed by the geotechnical consultant during excavation. This is to document the suitability of the exposed soils and to make necessary revisions if widely variable conditions are encountered. Revisions could include adding additional pits or a redesign of the system so that it conforms to the site conditions encountered during grading.

- Materials used in construction and installation of the infiltration systems should conform to the standards and specifications of the County and the State of California.

- The disposal of excessive turbid water or introduction of detergents and chemicals can cause premature system failures, necessitating construction of a system expansion or reconstruction of the primary system.

- Consideration should be given to perform a confirmatory infiltration test, following the construction of the basin. Typically, during a confirmatory infiltration test, the infiltration basin will be filled at an anticipated maximum flow rate for at least 100 minutes. The rate of infiltration of water into the subsurface soil should be recorded and documented. The test should be repeated about three times to establish the time interval required in between two consecutive discharge cycles.

5.3.2 Site Clearing

Any significant vegetation within the areas of proposed grading and construction should be stripped and removed from the site. Any deleterious construction debris (concrete, wood, sand bags, etc.) that is found to be existing on the surface of the site should also be removed.

All active or inactive utilities within the construction limits should be identified for relocation, abandonment, or protection prior to grading. Any pipelines greater than 4 inches in diameter to be abandoned in-place should be filled with a sand /cement slurry after review of their location and approval by the geotechnical engineer.

5.3.3 Ground Preparation – Improvement Areas

Based on field observations and laboratory test results, removal depths on the order of 2 feet below the subgrades of the footprint of the fill areas, access ramps, emergency overflow spillway and earthen swale improvement areas should generally be anticipated. Further, removal depths of 2 feet beneath the basin embankments should generally be anticipated. The removal and compaction of fill should extend at least 2 feet beyond the exterior limits of the improvements, discussed above. The removal and compaction of fill should also extend at least 2 feet beyond the interior limits of the basin embankments. The depth/zone of over-excavation may be larger if unsuitable materials are encountered during grading.

The removal area may then be restored to proposed grade with compacted fill (import or native, as described in Section 5.3.5).

5.3.4 Ground Preparation – Slope Facing

If shotcrete facing is planned for the side slopes of the infiltration basin, it is recommended that the side slopes have a slope ratio no steeper than 3:1 (horizontal to vertical). If shotcrete is used, a toe down with a minimum depth of 1 foot below planned grade should be considered to resist undercutting. Based on site-borings, these cuts would expose loose, low density Silty Sand to Poorly Graded Sand materials and may not provide a competent subgrade for shotcrete concrete. In order to mitigate the detrimental effects of differential settlements of these low-density materials on the shotcrete, we recommend construction of a 5-foot wide Fill Key at the toe of the slope. The Fill Key should be seated a minimum of 24 inches into the competent material and be tilted back into native alluvial soils at a minimum of 2 percent gradient. The back cut of the Fill Key may be benched at an equivalent slope angle of 45 degrees.

5.3.5 Fills/Backfills and Compaction

Onsite materials are generally considered suitable to be used as compacted fill, provided they meet the requirements of Section 5.2.5.

Prior to replacing the over-excavated soils or placing the import soils as properly compacted fill, the exposed bottom surfaces should first be scarified to a depth of 6 inches, watered or dried as necessary to achieve a uniform water content that is equal to or slightly greater than OWC, and then re-compacted in place to a minimum relative compaction of 90 percent. This procedure should be followed in areas of new fill, in areas to remain at existing grade, and in shallow cut areas where the depth of cut is less than 2 feet.

The embankment fills should be moisture conditioned to above OWC and placed in lifts no greater than 8 inches. Relative compaction of 90 percent minimum in accordance with ASTM D1557 is recommended for all fill embankments.

Placement of shotcrete on the slope face should be performed with care so as not to damage the slope face. Due to the desert region with extreme temperatures, placement and curing of concrete for the facing should be performed in such manner that the extreme temperatures and low humidity do not affect the curing process of the facing. Too hot or too cold temperatures will impact the shotcrete placement/curing and generate undesirable cracking of the shotcrete facing.

5.3.6 Imported Soils

Based on our current understanding of the project, excess soil materials will be generated due to the proposed grading operations and therefore soil needs to be exported offsite.

However, if imported soils are required to complete the planned grading, the soils should consist of clean materials devoid of rock exceeding a maximum dimension of 8 inches, as well as organics, trash and similar deleterious materials. Imported soils should also exhibit an expansion index of less than 20. If import soils are required, the project geotechnical consultant should be notified of the location of the proposed borrow site so that samples of the import material may be obtained and tested prior to transport to verify that it meets project geotechnical specifications.

5.3.7 Geotechnical Observations

Observations of the clearing operations, removal of surficial soils and general grading procedures should be performed by a representative of the project geotechnical consultant. It is the grading contractor's responsibility to notify the project geotechnical consultant at least one full workday (24 hours not including weekend days and holidays) prior to requiring observation (including excavation bottom verification). A representative of the project geotechnical consultant should be present on site during major grading operations to document that proper placement and adequate compaction of fills has been achieved, as well as to observe compliance with the other recommendations presented herein.

5.4 Foundation Design

Foundation recommendations provided below should not be modified without the geotechnical engineer’s review. Recommendations for slab-on-grade are included in Section 5.4.5 of this report.

5.4.1 Allowable Bearing Pressures

Lightly loaded facilities or structures can be founded on shallow footings. For design purposes, an allowable bearing pressure of **2,000 pounds per square foot** (psf) may be used for shallow footings (including spread and continuous footings) founded entirely in properly conditioned and compacted Structural Fill. The Structural Fill pad should extend at least 3 feet below the bottom of the footings and 5 feet outside the footings. Shallow footings designed for the bearing value recommended above should have a minimum width of 24 inches. Footings should be embedded at least 24 inches below the lowest adjacent finished grade. As stated before, due to the presence of loose soils at the anticipated bottom of footing elevation, it is recommended that native soils within 3 feet from the bottom of foundation or slab on grade be removed and replaced with structural backfill following recommendations in this technical memorandum. It is expected that over-excavation to a depth of 3 feet will expose firm and unyielding surface below the planned bottom of excavation or base of fill. If firm surface is not encountered at that depth, it is recommended to compact the native material in-place prior to placing compacted fill. The compacted fill should extend a minimum of 5 feet beyond the edges of foundation.

Shallow foundations are proposed for the project site. It is anticipated that all structures will be founded on mat foundations or slab-on-grade. If a mat is being considered for providing foundation support for the proposed facilities, the mat should be founded on a minimum 3-foot thick layer of compacted Structural Fill (over-excavation requirement). A maximum allowable bearing pressure of **2,000 psf** may be used for mat foundations. The bearing capacity of the foundation is limited by settlement. A value of k_s (modulus of subgrade reaction) of **150 pounds per cubic inch** (pci) may be used for design of a rectangular mat foundation with dimensions of 40 x 100 feet, where the k_s value was estimated on the basis of a common correlation between soil type and relative density. It is noted that a k_s value is typically derived from the results of a 1-foot by 1-foot square plate load test. Mat foundations designed for the bearing value recommended above should be embedded at least 24 inches below lowest adjacent finished grade. If the dimensions of the mat foundation are changed, the project geotechnical engineer should be consulted.

No structure foundations should bear partially on cut materials and partially on fill materials. In accordance with the recommendations in Section 5.2.4, Excavation, the upper soils native soils would be removed and replaced with Structural Fill, so all foundations would bear directly on fill mat. It is also possible that all the structure’s foundations could bear directly on native soil, provided the all excavations extend below the soil native soils found in the upper 3 feet or so of the site.

If the construction of the footings is not performed immediately after completion of grading, the near surface soils should be re-evaluated and approved by the geotechnical engineer-of-record immediately prior to placement of concrete for the proposed foundation.

5.4.2 Settlement

Based on the allowable bearing pressures and the earthwork recommendations presented in this report, total post-construction settlement of shallow footings or mat foundations is estimated to be less than or equal to about one inch. Differential settlements between similarly loaded footings designed for the bearing values recommended in this report are expected to be less than one-half the total settlement.

5.4.3 Resistance to Lateral Loads

Lateral forces applied to a structure will be resisted by either passive soil resistance against the buried part of the foundation or by sliding friction between the footing and the subgrade. We recommend that if sliding friction and passive soil resistance are combined, passive resistance should be reduced by one-third to account for the difference in the movements required to reach peak resistance.

For design purposes, an ultimate coefficient of friction of 0.4 may be used for footings cast on properly conditioned and compacted subgrade. Ultimate passive pressure available in compacted structural fill may be taken as equivalent to the pressure exerted by a fluid weighing 360 pcf per foot (psf/ft) of depth with a maximum limiting value of 2000 psf (use 180 psf/ft up to a maximum limiting value of 1000 psf if below groundwater). The pressure should be used as a triangular distribution to the maximum allowable limit and then should remain constant at the maximum limiting value. If the ground surface is not covered by permanent concrete slab-on-grade or asphalt pavement, the effective ground surface should be taken as 12 inches lower than the actual post-construction ground surface for the purpose of calculating the passive soil resistance. Appropriate factors of safety should be applied to the above values of ultimate resistances.

5.4.4 Foundation Design Parameters

Bearing Material: 3 feet of structural fill over native soil

Foundation Design Parameters:

Minimum Footing Depth:	24 inches below lowest adjacent final grade
Allowable Bearing Pressure:	2,000 psf
Coefficient of Vertical Subgrade Reaction:	150 pounds per cubic inch (pci)
Coefficient of Sliding:	0.4
Slab Thickness:	Per structural engineer
Slab Subgrade Water Content:	OWC to OWC plus 3%
Cement Type:	I or II
Steel Reinforcement Cover:	Minimum concrete cover of 3 inches
Ultimate Passive Resistance:	360 psf/ft up to a maximum of 2000 psf (No increase for short-term loads; disregard upper 12 inches of ground unless paved; when combined with frictional resistance, passive resistance should be reduced by one-third)
Vapor Retarder:	Stego 15 mil Class A or equivalent No sand required beneath vapor retarder Sand above retarder - per structural engineer

5.4.5 Slab On grade

Conventional concrete slab-on-grade floors may be used for the proposed structures. The slab thickness and reinforcement should be designed by the structural engineer for the anticipated floor loads and other structural considerations. These floors should be supported on a pad of compacted Structural Fill. The Structural Fill pad should extend at least 2 feet below bottom of floor slabs, drainage blanket, or thickened slab edges.

Any materials disturbed during construction should be removed and replaced with Structural Fill properly moisture conditioned and compacted to at least 95 percent relative compaction. The water content of subgrade soil should be maintained at a level slightly over its optimum water content until the slab is poured. At the time of concrete placement, the subgrade soil should be firm and relatively unyielding.
If a moisture-sensitive floor covering (such as tile) is planned in any of the structures, the floor slab should be underlain by an impermeable polyethylene membrane, at least 15-mills thick, covered with a two-inch layer of moistened (not saturated) clean sand (less than 5 percent of particles passing the No. 200 sieve) to both protect the membrane and to promote concrete curing. It may also be prudent to provide a thin layer of clean, coarse sand beneath the membrane to act as a capillary break and to protect the membrane from the underlying subgrade materials.

5.4.6 Pavement Recommendations

Pavement design analyses were based on the California Highway Design Manual (Caltrans, 2016a). In this method, soil and base material strengths are evaluated with respect to an R-value and traffic information is estimated in the form of a traffic index (TI). The exposed subgrade soils should be scarified to a depth of 6 inches; moisture conditioned to not less than the OWC, and compacted to at least 90% relative compaction as determined by ASTM Test Method D1557.

- Either Caltrans Class 2 aggregate base (AB) or an similar material such as Crushed Miscellaneous Base (CMB) should be utilized for the AB section and should be moisture conditioned to at least its OWC and compacted to at least 95% relative compaction.
- The planned hot mix asphalt (HMA) portion of the pavement section should be placed in loose lifts of 4 inches maximum in thickness, compacted and tested per California Test Method 375. The type of AC should consider the hot climate and extreme temperature range and meet the minimum standards set forth by City of Desert Hot Springs or local jurisdiction.
- At this time traffic information is not available. Flexible pavement recommendations for a 20-year design life were calculated using Caltrans' computer program CalFP version 1.5 (Caltrans, 2016b) and are included in Appendix D of this report and a summary of the results is presented in **Table 9** below:

Table 9 – Pavement Design Summary

	Traffic Index	Minimum Thickness HMA (inch)	Minimum Thickness AB (inch)
R-value = 50	**TI=5**	4	4.5
	TI=6	5	4.5
	TI=7	6	4.5

* HMA = Hot mix Asphalt, AB= Aggregate Base

Section 6 – Notes to Designer

6.1 Review of Plans and Specifications

Final project plans and specifications should be reviewed by the geotechnical engineer-of-record prior to construction to confirm that the full intent of the recommendations presented in this report has been applied to the design and that the recommendations presented are applicable to the final scope of the project.

Section 7 – Limitations

This memorandum has been prepared for Mission Springs Water District's use for the project described herein only, and is not to be distributed to or used by third parties without the written consent of AECOM.

AECOM has observed only a small portion of the pertinent subsurface conditions. The recommendations made in this report are based on the assumption that soil and geologic conditions do not deviate appreciably from those observed in the subsurface explorations. The project quality control should provide observation and testing during foundation excavation, fill placement, and other forms of construction that need geotechnical input to evaluate whether the site conditions are as anticipated, and to provide revised recommendations, if necessary. If variations or undesirable geotechnical conditions are encountered during construction, the geotechnical engineer-of-record should be consulted for further recommendations.

Geotechnical engineering and the geologic sciences are characterized by uncertainty. Professional judgments presented herein are based on the assumption that subsurface conditions do not vary significantly between borings, or vary linearly between borings. The recommendations provided in this report also are based partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgments rendered meet current professional standards; we do not guarantee the performance of the project in any respect.

Section 8 – References

California Building Standards Commission, 2016, California Building Code: California Code of Regulations, Title 24, Part 2, Volumes 1 and 2, based on the 2015 International Building Code.

California Division of Mines and Geology (CDMG, 1980). California Division of Mines and Geology, State of California, Special Studies Zones, Desert Hot Springs Quadrangle. January 1, 1980.

California Division of Mines and Geology (CDMG, 1965). Geologic Atlas of California, Map No. 019, 1:250,000 scale. Complied by Thomas H. Rogers. Dated 1965.

California Geological Survey (CGS, 2010). Fault Activity Map of California. Retrieved from http://maps.conservation.ca.gov/cgs/fam/

California Geological Survey (CGS, 2012). Preliminary Geologic Map of Quaternary Surficial Deposits in Southern California, Palm Springs, 30'x60' Quadrangle, Version 1.0, Compiled by J.T. Lancaster, C.A. Hayhurst & T.L. Bedrossian.

Caltrans (2010), Soil and Rock Logging Classification Manual, Division of Engineering Geotechnical Services, 2010.

Caltrans (2015). Corrosion Guidelines, Version 2.0. Division of Engineering Services Materials Engineering and Testing Service Corrosion Technology Branch, November 2015.

Caltrans (2016a), Highway Design Manual*,* 2016.

Caltrans (2016b), Computer Program CalFP Version 1.5, A Windows-based program to design new pavement alternatives for flexible pavements*,* Accessed October 2017.

Caltrans (2017a). Caltrans ARS Online (Version 2.3.09) Website: http://dap3.dot.ca.gov/ARS_Online/.

GeoLogic Associates [GLA] (2004). "Geotechnical Report, Garnet Well Suction Reservoir, Mission Springs Water District, Desert Hot Springs, Riverside County, California." GeoLogic Associates, dated September, 2004. Prepared for RW Beck.

Landmark Consultants, Inc. [LCI] (2008). "Geotechnical Investigation Report, Proposed Future Regional Wastewater Facility, Desert Hot Springs, California." LandMark Consultants, dated April, 2008. Prepared for Mission Springs Water District.

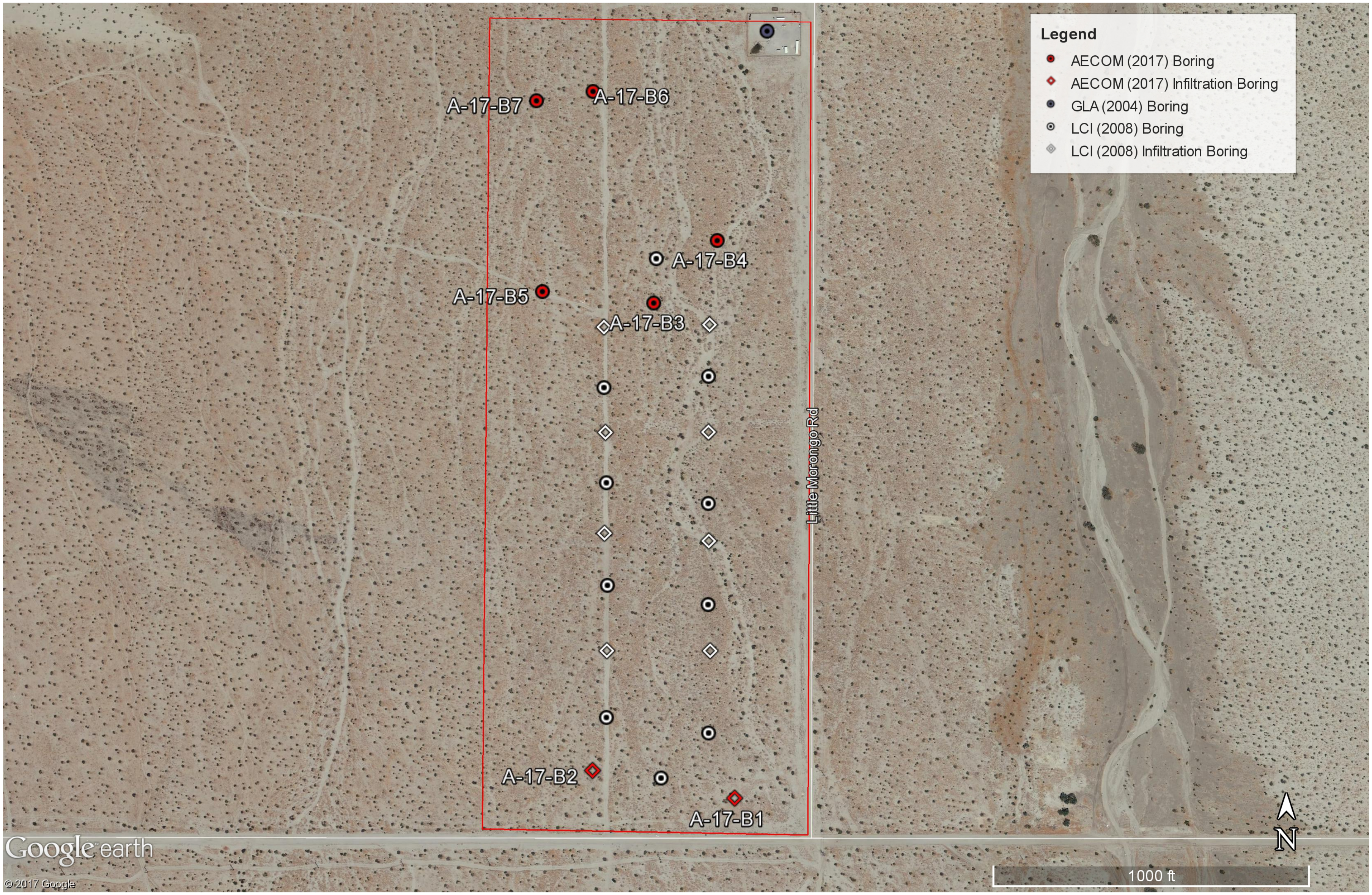
Montgomery Watson Harza Global (MWH, 2013). "Mission Creek / Garnet Hill Water Management Plan, Final Report." MWH, dated January, 2013. Prepared for Coachella Valley Water District, Desert Water Agency and Mission Springs Water District.

Riverside County Flood Control and Water Conservation District (RCFCWCD, 2011). "Design Handbook for Low Impact Development Best Management Practices for the Santa Ana Watershed." Retrieved from http://www.floodcontrol.co.riverside.ca.us/NPDES/LIDBMP.aspx

Figures

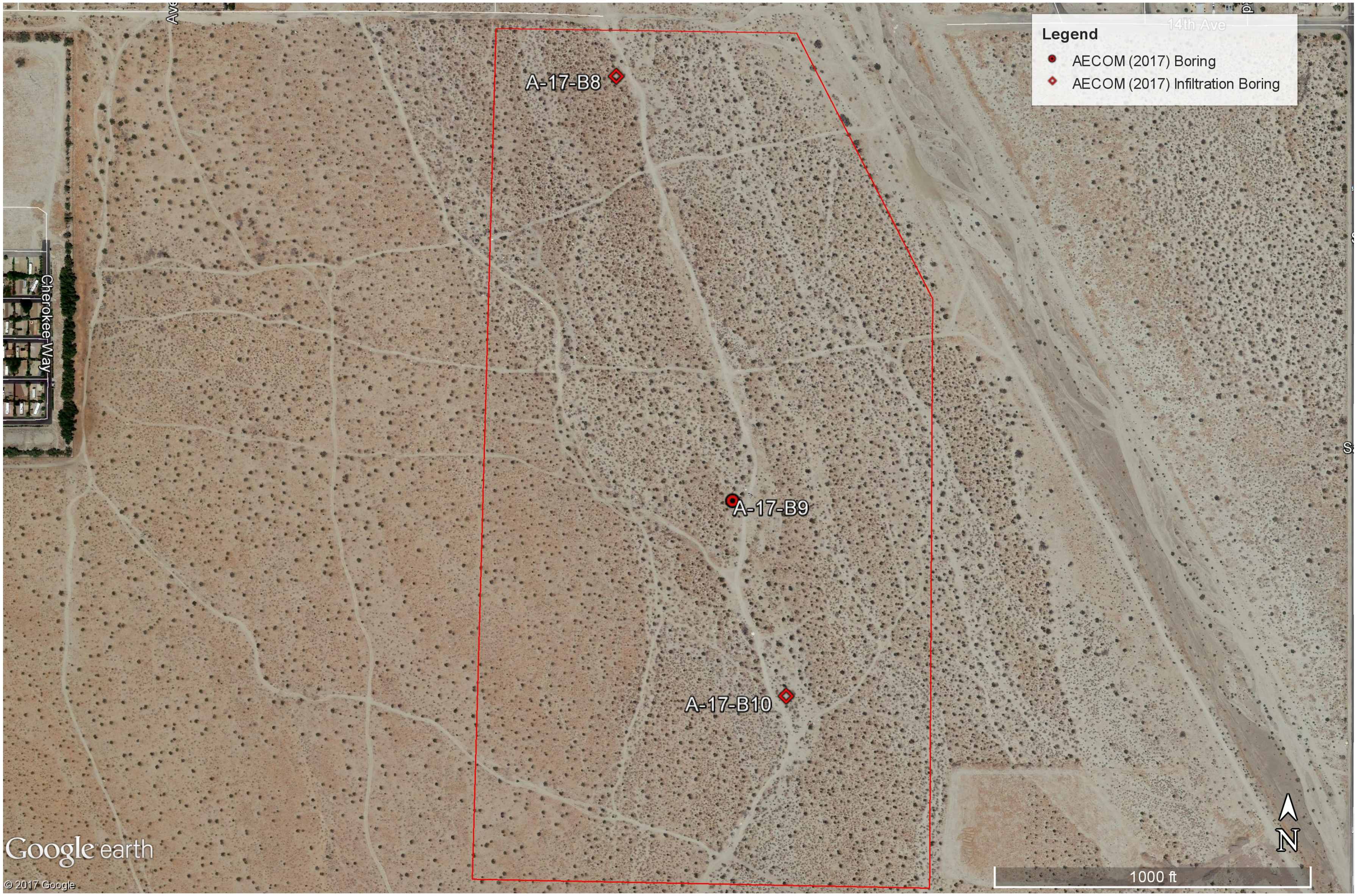
Site Location Map

Project No. 60551186	Date: Oct. 2018	Project: Mission Springs Water District WVWRF	Figure 1	AECOM



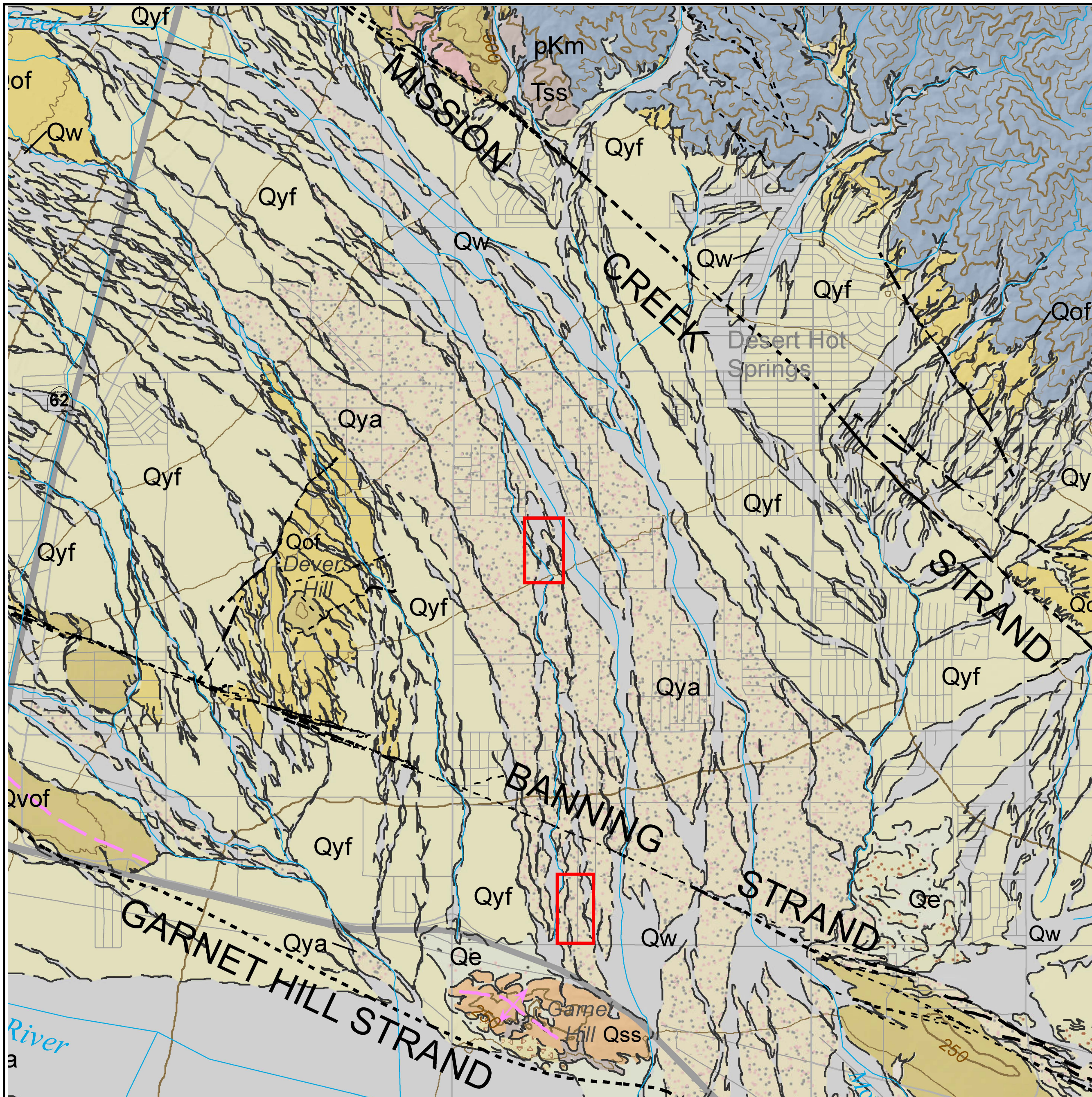
Geotechnical Field Exploration Map - Mission Springs WWTP

Project No. 60551186	Date: Oct. 2018	Project: Mission Springs Water District WVWRF	Figure 2a	AECOM



Geotechnical Field Exploration Map - Off-site Spreading Basin

Project No. 60551186	Date: Oct. 2018	Project: Mission Springs Water District WVWRF	Figure 2b	AECOM



Source: *California Geologic Survey, 2012 Preliminary Geologic Map of Quaternary Surficial Deposits in Southern California, Palm Springs, 30' x 60' Quadrangle, Version 1.0, Compiled by J.T. Lancaster, C.A. Hayhurst & T.L. Bedrossian*

0 1 2

Scale in Miles

N

LEGEND

Qw – Alluvial wash deposits (Late Holocene)
Qe – Eolian and dune deposits (Late Holocene)
Qya – Young alluvial valley deposits (Holocene to Late Pleistocene)
Qyf – Young alluvial fan deposits (Holocene to Late Pleistocene)
Qof – Old alluvial fan deposits (Late to Middle Pleistocene)
Qvof – Very old alluvial fan deposits (Middle to Late Pleistocene)
Qss – Coarse-grained sandstone bedrock (Quaternary)
Tss – Sandstone Bedrock (Tertiary)
Pkm – Cretaceous and Pre-Cretaceous Metamorphic (sedimentary and volcanic)

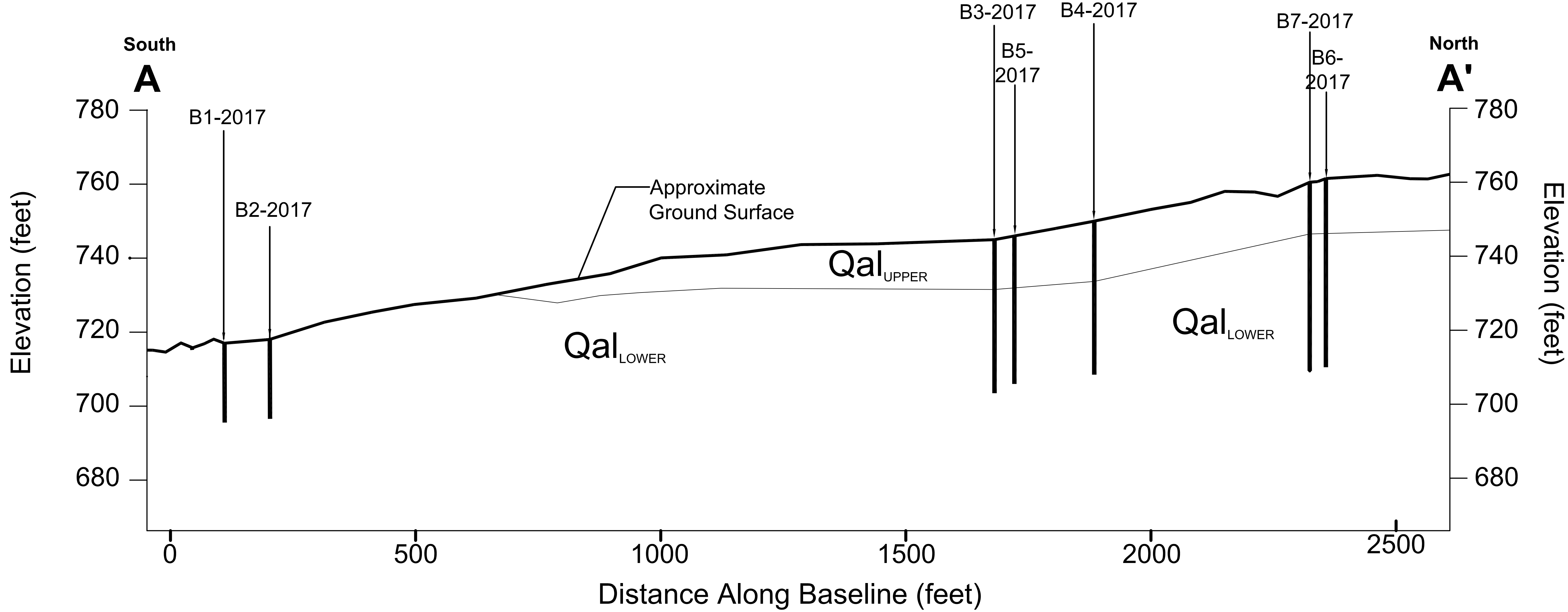
REGIONAL GEOLOGIC MAP

Project No.: 60551186	Date: Oct. 2018	Project: Mission Springs Water District WVWRF	Figure 3

AECOM

GENERALIZED GEOLOGIC PROFILE PRIMARY WVWRF

SITE VIEW WEST



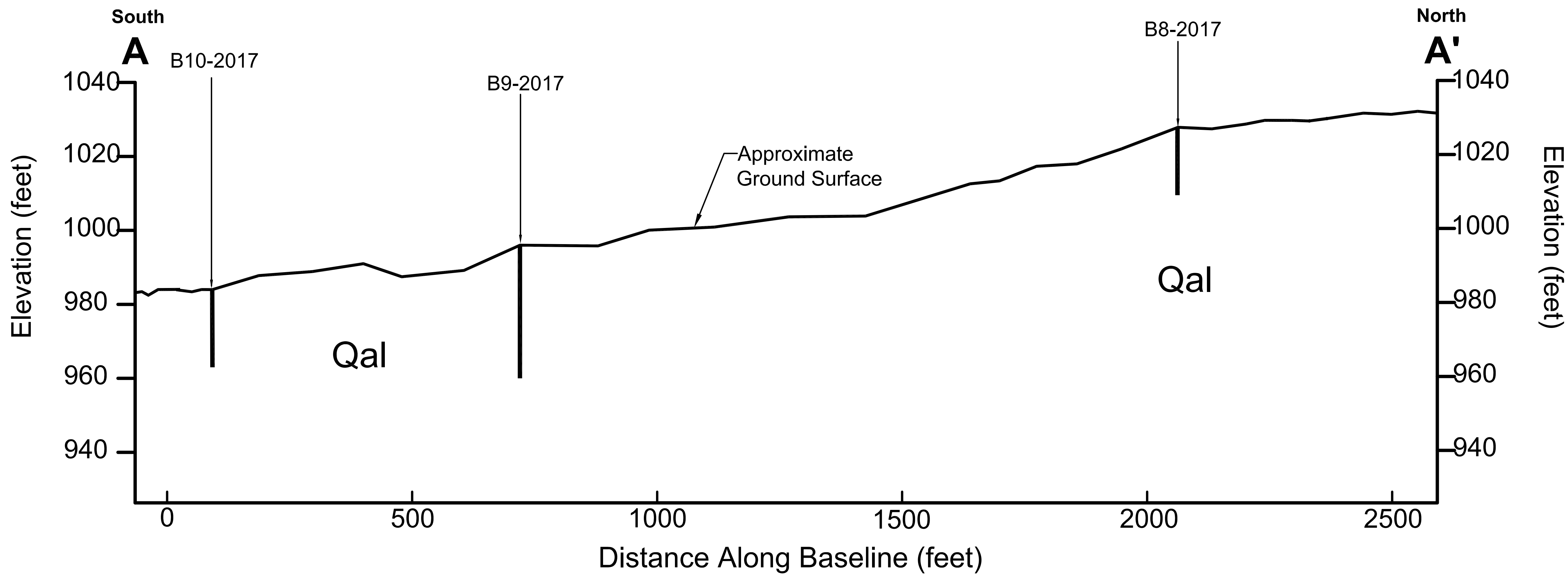
LEGEND

Qal Quaternary Alluvium

NOT TO SCALE

GENERALIZED GEOLOGIC PROFILE PRIMARY WVWRF	
Proj. No. 60551186	Date: 10-24-2018
Project: Mission Springs Water District WVWRF	Figure 4A

GENERALIZED GEOLOGIC PROFILE ALTERNATIVE
OFF-SITE SPREADING BASIN
SITE VIEW WEST



LEGEND

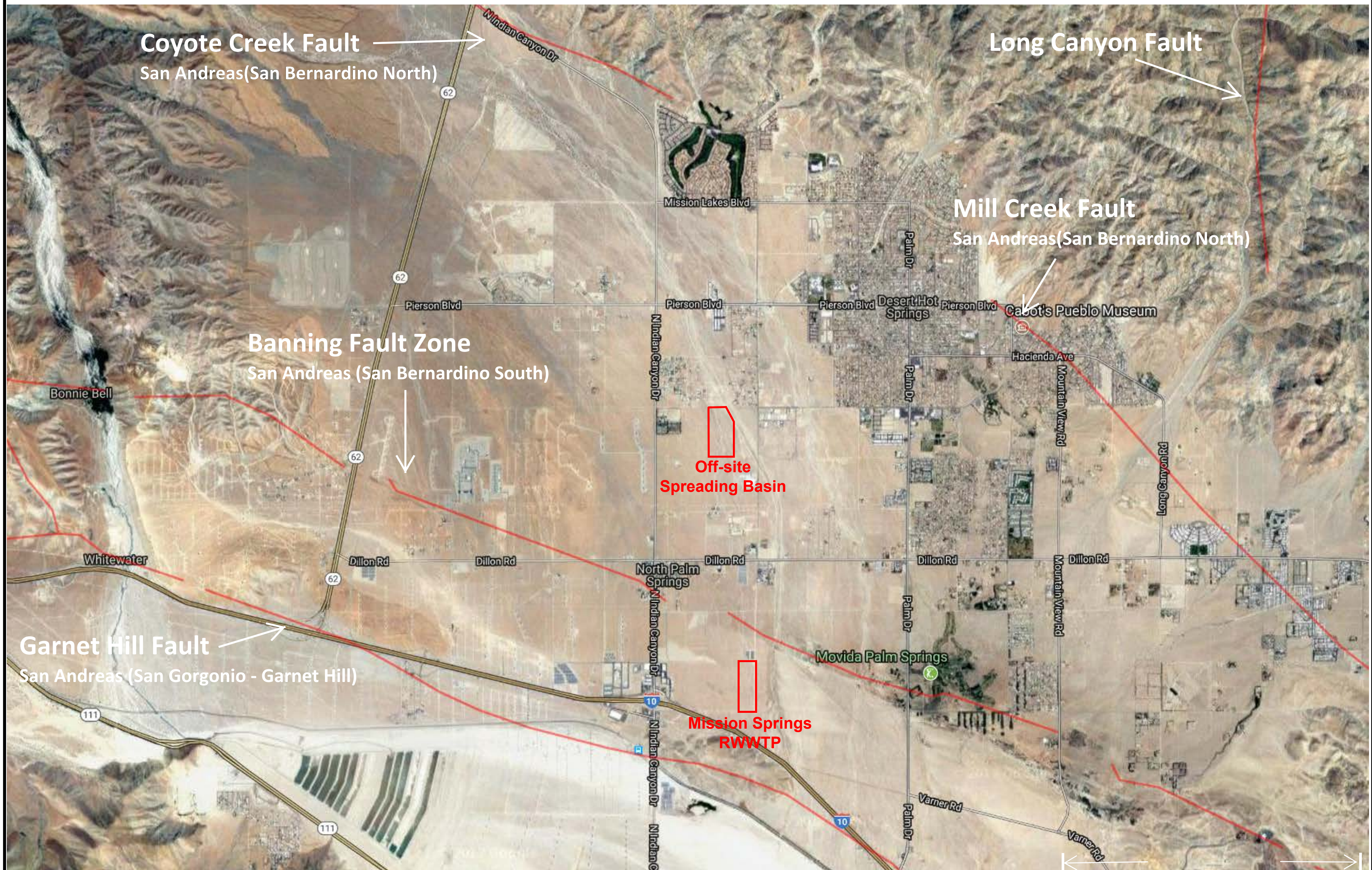
Qal Quaternary Alluvium

NOT TO SCALE

AECOM

GEOLOGICAL CROSS SECTION
A—A'

Proj. No. 60551186	Date: 11-10-2017
Project: Mission Springs Water District RWWTP	Figure 4B



FAULT MAP

Project No. 60551186	Date: Oct. 2018	Project: Mission Springs Water District WVWRF	Figure 5	AECOM

Appendix A

Field Boring Logs

A geotechnical field exploration was performed between September 26th and October 4th, 2017 under the supervision of AECOM. A site reconnaissance was performed by an AECOM engineer/geologist prior to the field exploration to identify locations of exploratory borings. The locations were located in the field from the existing site features. AECOM notified Underground Service Alert (USA) so that they could coordinate with various utility companies to locate and clear existing underground lines in the vicinity of the planned exploration.

Subsurface exploration included drilling and sampling 10 hollow stem auger borings in the area of the proposed RWWTP project. The borings were drilled to a maximum depth of approximately 50 feet below ground surface (bgs) using a truck-mounted CME-75 drill rig with an attached CME Auto Hammer. The drill rigs were provided and operated by 2R Drilling of Chino, California. The approximate locations of the borings are shown on Table 1.

An AECOM geotechnical representative was tasked with maintaining field boring logs and visually classified the soils according to the Caltrans Soil and Rock Logging Classification and Presentation Manual (Caltrans, 2010). When subsurface conditions permitted, drive samples were recovered with the California Soil Sampler [(2.42-inch) I.D.] and disturbed samples were obtained using a Standard Penetration Testing (SPT) sampler. The samples were obtained using a 140-pound automatic-trip hammer with a 30-inch drop. The number of blows required to drive the sampler was recorded at 6-inch intervals for each sample taken. SPT was performed in accordance with ASTM D1589 procedures. The total number of blows required to drive the sampler the last 12 inches is recorded on boring records.

Geotechnical samples obtained in the field were carefully sealed and packaged to reduce moisture loss and disturbance and were transported to our laboratory for further testing. After completion of drilling and sampling operations, borings were backfilled with cement/bentonite slurry.

The blow count for the final 12 inches of sampler penetration is commonly referred to as the "N-value". This value generally reflects the resistance to penetration of the soil at the sample depth. The degree of relative density of granular soils and the degree of consistency of cohesive soils are generally described on the boring logs according to the conventional correlations presented below:

Granular Soils			Cohesive Soils	
SPT Blow Count	**Description**		**Pocket Penetrometer Measurement, PP (tsf)**	**Description**
$N_{60} \leq 4$	Very Loose		PP < 0.25	Very Soft
$5 \leq N_{60} \leq 10$	Loose		0.25 ≤ PP < 0.5	Soft
$11 \leq N_{60} \leq 30$	Medium Dense		0.5 ≤ PP < 1	Medium Stiff
$31 \leq N_{60} \leq 50$	Dense		1 ≤ PP < 2	Stiff
$50 < N_{60}$	Very Dense		2 ≤ PP < 4	Very Stiff
			4 ≤ PP	Hard

The relative density and consistency descriptions on the attached boring logs are based on adjusted blow counts recorded in the field. These numbers are considered useful in providing an estimate of the relative density or consistency of soils. The relative density and consistency descriptions on the log may deviate from the correlation for a number of reasons, including reliance on other test results or the engineer's judgment based on manual manipulation of the sample.

It is widely accepted that the above-listed SPT blow count correlation is overly simplistic. For most applications in non-gravelly soils, the blow count is usually adjusted for the effective vertical pressure at the sampling depth and for other sampling system parameters such as the efficiency of the sampling system and/or sampling techniques used. In gravelly soils, it is recognized that the blow counts are higher than would be expected in non-gravelly soils of similar density or consistency. This occurs because the sampler tends to push larger gravel clasts ahead of it. The area of the gravel clast may be significantly greater than that of the sampler, causing increased resistance and higher blow counts.

The blow count obtained from nonstandard penetration tests using a California Soil Sampler, N, may be converted to standard blow count, N_{60}, by the relationship between SPT values and hammer ratios, Rs = f(inner/outer diameter of sampler, weight of hammer, and height of drop), (Fang, 1991). The conversion factors for California Sampler blow counts used for sandy soil are 0.55 and 0.70 for cohesive soil, respectively. An energy efficiency correction factor of 1.345 (ERi = 80.7%) was applied to correct blow counts for the borings A-17-B1 to A-17-B10.

SPT HAMMER ENERGY MEASUREMENTS

Prepared by;

SPT CAL
5512 Belem Dr
Chino Hills, CA 91709

909-730-2161
bc@sptcal.com

2R Drilling, Inc.
3968 Chino Ave.
Chino, CA 91710
909-465-1765

Project Title: 2R Drilling Rig 7 2017
Project Description: Ontario

Rig 7

Energy Transfer Ratio = 80.7 @ 54.1 blows per minute

Testing was performed on July 12, 2017 in Ontario, California

Hammer Energy Measurements performed in accordance to ASTM D4633 using an approved and calibrated SPT Analyzer from Pile Dynamics, Inc.

Depth	ETR%	BPM
30	80.0	53.9
35	81.1	54.5
40	81.9	54.0
45	80.2	54.4
50	80.3	53.9
	80.7	54.1

Thank you very much. It was a pleasure to work with you and your drill crews.

Sincerely yours,

Brian Serl
Calibration Engineer
SPTCAL.COM

PRESENTATION OF SPT ANALYZER TEST DATA

1. Introduction

This report presents the results of SPT Hammer Energy Measurements recorded with an SPT Analyzer from Pile Dynamics carried out on July 12, 2017 in Ontario, California

2. Field Equipment and Procedures

The drill used is referred to at 2R Drilling as Rig 7. CME 75 track drill. It has an attached CME Auto Hammer

The CME Auto Hammer uses a 140 lb. weight dropped 30” on to an anvil above the bore hole. AWJ drill rod connects the anvil to a split spoon type soil sampler inside an 8” o.d. hollow stem auger at the designated sample depth. After a seeding blow the sampler is driven 18”. The number of blows required to penetrate the last 12" is referred to as the “N value”, which is related to soil strength.

The first recording was taken at 30' below ground surface and then every 5' to final recording at 50'.

3. Instrumentation

An SPT Analyzer from Pile Dynamics was used to record and the process the data. The raw data was stored directly in the SPT Analyzer computer with subsequent analysis in the office with PDA-W and PDIPlot software. The measurements and analysis were conducted in general accordance with ASTM D4945 and ASTM D6066 test standards.

The SPT Analyzer is fully compliant with the minimum digital sampling frequency requirements of ASTM D4633-05 (50 kHz) and EN ISO 22476-3:2005 (100 kHz), as well as with the low pass filter, (cutoff frequency of 5000 Hz instead of 3000 Hz) requirements of ASTM D4633-05. All equipment and analysis also conform to ASTM D6066.

A 2' instrumented section of AWJ rod, with two sets of accelerometers and strain transducers mounted on opposite sides of the drill rod, was placed below the anvil. It measured strain and acceleration of every hammer blow. The SPT Analyzer then calculates the amount of energy transferred to the rod by force and velocity measurements.

4. Observations

The drill rig motor is diesel fueled. The throttle control is electronically controlled. The per minute average was very consistent for every interval. The drill and sample equipment looked well maintained and operated

5. Results

Results from the SPT Hammer Energy Measurements are summarized below. It shows the Energy Transfer Ratio (ETR) at each sampling depth. ETR is the ratio of the measured maximum transferred energy to rated energy of the hammer which is the product of the weight of the hammer times the height of the fall. 140 lb x 30” = 4200 lb-in = 0.350 kip-ft.

Energy Transfer Ratio = 80.7 @ 54.1 blows per minute

Depth	ETR%	BPM
30	80.0	53.9
35	81.1	54.5
40	81.9	54.0
45	80.2	54.4
50	80.3	53.9
	80.7	54.1

If you have any questions please do not hesitate to call or email.

Thank you,

Brian Serl
Calibration Engineer
SPT CAL
909-730-2161
bc@sptcal.com

Project: MSWD-Regional WWTP Project Location: North Palm Springs, CA Project Number: 60551186 1.13	**Key to Logs** Sheet 1 of 1

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Well constructed	Water Content, %	Dry Density,pcf	REMARKS AND OTHER TESTS
1	2	3	4	5	6	7	8	9	10	11

COLUMN DESCRIPTIONS

1 **Elevation:** Elevation in feet referenced to mean sea level (MSL) or site datum.

2 **Depth:** Depth in feet below the ground surface.

3 **Sample Type:** Type of soil sample collected at depth interval shown; sampler symbols are explained below.

4 **Sample Number:** Sample identification number.

5 **Blows per foot** Number of blows required to advance driven sampler each 6-inch drive interval, or distance noted, using a 140-lb hammer with a 30-inch drop.

6 **Graphic Log:** Graphic depiction of subsurface material encountered; typical symbols are explained below.

7 **Material Description:** Description of material encountered; may include relative density / consistency, moisture, color, and grain size.

8 **Well constructed:** Graphic depiction of piezometer or well installation; materials are listed in header block; graphic symbols are explained below.

9 **Water Content:** Water content of soil sample measured in laboratory, expressed as percentage of dry weight of specimen.

10 **Dry Unit Weight:** Dry density of soil sample measured in laboratory, in pounds per cubic foot.

11 **Remarks and Other Tests:** Comments and observations regarding drilling or sampling made by driller or field personnel. Other field and laboratory test results, using the following abbreviations:

PA	Sieve Analysis (%<#200 sieve)
WA	Wash Analysis (%<#200 sieve)
LL	Liquid Limit, from Atterberg limits test (%)
PI	Plasticity Index (LL-PL) (%)
DS	Direct Shear test
CU	Consolidated-Undrained Triaxial

TYPICAL SOIL GRAPHIC SYMBOLS

- Asphalt Concrete
- Aggregate Base
- SILTY SAND (SM)
- SILTY SAND with GRAVEL (SM)
- CLAYEY SAND (SC)
- CLAYEY SAND with GRAVEL (SC)
- CLAYEY GRAVEL (GC)
- Poorly graded GRAVEL with SILT and SAND (GP-GM)

TYPICAL WELL GRAPHIC SYMBOLS

- Pipe top cap, inside 12" flush-mount well cover, set in concrete
- 2" blank PVC (Schedule 40) inside flush-mount well cover, set in concrete
- 2" blank PVC (Schedule 40) in concrete
- 2" blank PVC (Schedule 40) in cement/bentonite grout
- 2" blank PVC (Schedule 40) in bentonite chips
- 2" blank PVC (Schedule 40) in #2/12 clean graded sand
- 2" screened PVC (Schedule 40) in #2/12 clean graded sand
- Pipe end cap, in #2/12 clean graded sand
- Borehole backfill, #2/12 clean graded sand

TYPICAL SAMPLER GRAPHIC SYMBOLS

- Bucket or grab sample
- Modified California sampler
- Standard Penetration sampler

OTHER GRAPHIC SYMBOLS

- ▽ First water encountered at time of drilling and sampling
- ▼ Water level measured at specified time after completion of drilling and sampling
- —— Contact between strata
- — — Inferred or gradational contact between strata

GENERAL NOTES

1. Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive; actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of laboratory tests.
2. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.
3. All wells enclosed in 12 inch flush-mount well cover

Report: GEO_SOIL_WELL_KEY; File: BORINGS_B-1_B-10.GPJ; 11/13/2017 Key

Project: MSWD-Regional WWTP
Project Location: North Palm Springs, CA
Project Number: 60551186 1.13

Log of Boring A-17-B1

Date(s) Drilled	**09-27-17**	Logged By	**J. Leiva**	Checked By	**L. Vazquez**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" bullet bit**	Total Depth of Borehole	**21.5 feet**
Drill Rig Type	**Limited Acess Rig CME**	Drilling Contractor	**2R Drilling**	Approximate Surface Elevation	**714 feet**
Water Level Depth (Feet)	**Not Encountered**	Sampling Method(s)	**Bulk, SPT (1.4" I.D.)**	Hammer Data	**Automatic Hammer, 140 lbs / 30" drop**
Borehole Backfill	**Temporary well installed for infiltration testing**	Location	**33.903680° N -116.528750° W**	Hammer Efficiency Rating (ERi)	**81** %

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Well constructed	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	0	Bulk	S01			Poorly graded SAND with SILT (SP-SM); yellowish brown; dry; mostly fine SAND; few SILT; trace fine GRAVEL				
715										
	5	SPT	S02	24		medium dense; light brownish gray		0		PA: 9.3%<#200 sieve, only 6" recorvered, rock at the bottom of barrel, moved 5' NW
710										
	10	SPT	S03	30		dense; becomes coarsed grained		1		
705										
	15	SPT	S04	23		Well graded SAND with SILT (SW-SM); medium dense; light brownish gray; dry; mostly fine SAND; few SILT; few fine GRAVEL, fractured rock		1		PA: 7.7%<#200 sieve
700										
	20	SPT	S05	26		Poorly graded SAND with SILT (SP-SM); medium dense; yellowish brown; moist; mostly fine and coarse SAND; few fine GRAVEL		1		
695						Total Depth = 21.5 feet Temporary monitoring well installed 9/27/17 Temporary monitoring well over drilled with 8" hollow stem auger on 10/2/17. PVC pipe and screen removed and backfilled with cement-bentonite slurry. Well Notes: 0'-4.5' Backfill: cement-bentonite slurry 4.5'-5.5' Backfill: Pel-Plug Pellets 5.5'-21.5 Backfill: Backfill: #2/12 Clean graded sand 2" I.D Solid PVC pipe 0'-9.5' 2" I.D. Schedule-40 slotted PVC (size 0.010); 9.5'-21'				
	25									
690										
	30									

Report: GEO_SOIL_WELL; File: BORINGS_B-1_B-10.GPJ; 11/13/2017 A-17-B1

Project: MSWD-Regional WWTP
Project Location: North Palm Springs, CA
Project Number: 60551186 1.13

Log of Boring A-17-B2

Date(s) Drilled	**9-27-17**	Logged By	**J. Leiva**	Checked By	**L. Vazquez**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" bullet bit**	Total Depth of Borehole	**21.5 feet**
Drill Rig Type	**Limited Acess Rig CME**	Drilling Contractor	**2R Drilling**	Approximate Surface Elevation	**715 feet**
Water Level Depth (Feet)	**Not Encountered**	Sampling Method(s)	**Bulk, SPT (1.4" I.D.), Mod CAL**	Hammer Data	**Automatic Hammer, 140 lbs / 30" drop**
Borehole Backfill	**Temporary well installed for infiltration testing**	Location	**33.903920° N -116.530230° W**	Hammer Efficiency Rating (ERi)	**81** %

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Well constructed	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	0					Poorly graded SAND (SP); yellowish brown; dry to moist; mostly fine SAND; trace SILT		1		
715			S01							
	5		S02	31		Poorly graded SAND with SILT (SP-SM); dense; pale yellowish brown; dry; mostly fine and coarse SAND; few SILT; trace fine GRAVEL		1	111	
710										
	10		S03	22		Well graded SAND with SILT (SW-SM); medium dense; light brownish gray; dry; mostly fine and coarse SAND; few SILT; trace coarse GRAVEL		1		PA: 5.7%<#200 sieve.
705										
	15		S04	50		Poorly graded SAND with SILT (SP-SM); very dense; light brownish gray; dry; moslty fine and coarse SAND; little SILT; few fine to coarse GRAVEL		1		WA: 8.6%<#200
700										
	20		S05	43		dense; few fine GRAVEL				fractured rock in barrel
695						Total Depth = 21.5 feet Temporary monitoring well installed 9/27/17 Temporary monitoring well over drilled with 8" hollow stem auger on 10/2/17. PVC pipe and screen removed and backfilled with cement-bentonite slurry.				
	25					Well Notes: 0'-5' Backfill: cement-bentonite slurry 5'-6' Backfill: Pel-Plug Pellets 6'-21.5 Backfill: Backfill: #2/12 Clean graded sand 2" I.D Solid PVC pipe 0'-9' 2" I.D. Schedule-40 slotted PVC (size 0.010); 9'-21'				
690										
	30									

Project: MSWD-Regional WWTP
Project Location: North Palm Springs, CA
Project Number: 60551186 1.13

Log of Boring A-17-B3

Date(s) Drilled	**9-28-17**	Logged By	**J. Leiva**	Checked By	**L. Vazquez**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" bullet bit**	Total Depth of Borehole	**41.5 feet**
Drill Rig Type	**Limited Acess Rig CME**	Drilling Contractor	**2R Drilling**	Surveyed Surface Elevation	**745 feet**
Water Level Depth (Feet)	**Not Encountered**	Sampling Method(s)	**Bulk, SPT (1.4" I.D.), Mod CAL**	Hammer Data	**Automatic Hammer, 140 lbs / 30" drop (Efficiency=81%)**
Borehole Backfill	**cement-bentonite slurry, covered with soil cuttings to match surface**	Location	**33.907970° N -116.529581° W**		

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
745	0		S01			Poorly graded SAND with SIILT (SP-SM); brownish gray; dry; mostly fine and little coarse SAND; few SILT; trace GRAVEL	0		
740	5		S02	33		Well graded SAND (SW); dense; light brownish gray; moslty fine and some coarse SAND; trace SIIT	1	116	PA: 4.6%<#200 sieve, DS
735	10		S03	20					Corr,
						SANDY SILT (ML); medium dense; dry; yellowish brown; some fine SAND			
730	15		S04	36		Poorly graded SAND (SP); dense; brownish gray; dry; mostly fine and coarse SAND; trace SILT			Dosturbed sample, loose sand, put in baggy, coarse Gravel in shoe
725	20		S05	17		Well graded SAND with SILT (SW-SM); medium dense; light brownish gray; dry; mostly medium and little fine SAND; trace GRAVEL	1		6.1%<#200 sieve.
720	25		NR	78		very dense			Coarse GRAVEL in shoe, No Recovery
715	30								

Project: MSWD-Regional WWTP
Project Location: North Palm Springs, CA
Project Number: 60551186 1.13

Log of Boring A-17-B3

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
715	30		S07	41		SILTY SAND (SM); dense; light olive brown; dry; mostly fine SAND; little SILT; trace fine GRAVEL	2		WA: 22.1%<#200, LL=21 PI=0
710	35		S08	60		very dense; brownish gray; mostly fine and some coarse SAND; little SILT;	1		fractured rock in barrel
705	40		S09	39		dense; becomes fined grained			
						Total Depth = 21.5 feet			
						Backfilled with cement-bentonite slurry			
700	45								
695	50								
690	55								
685	60								
680	65								

Report: GEO_10_SNA; File: BORINGS_B-1_B-10.GPJ; 11/13/2017 A-17-B3

Project: MSWD-Regional WWTP
Project Location: North Palm Springs, CA
Project Number: 60551186 1.13

Log of Boring A-17-B4

Date(s) Drilled	**9-28-17**	Logged By	**J. Leiva**	Checked By	**L. Vazquez**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" bullet bit**	Total Depth of Borehole	**32.0 feet**
Drill Rig Type	**Limited Acess Rig CME**	Drilling Contractor	**2R Drilling**	Surveyed Surface Elevation	**750 feet**
Water Level Depth (Feet)	**Not Encountered**	Sampling Method(s)	**Bulk, SPT (1.4" I.D.), Mod CAL**	Hammer Data	**Automatic Hammer, 140 lbs / 30" drop (Efficiency=81%)**
Borehole Backfill	**cement-bentonite slurry, covered with soil cuttings to match surface**	Location	**33.908510° N -116.528919° W**		

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
750	0		S01			Poorly graded SAND with SILT (SP-SM); pale brownish gray; dry; mostly fine and coarse SAND; few SILT			R-Value: 77
745	5		S02	37		Well graded SAND with SIILT (SW-SM); dense; grayish brown; dry mostly fine and coarse SAND; trace fines	1		PA: 7.7%<#200 sieve.
740	10		S03	50/5"		Poorly graded SAND with SILT (SP-SM); very dense; light brownish gray; few SILT; little fine and coarse GRAVEL	1		WA: 9.9%<#200, rock in shoe at 13'. Move 5' N
735	15		NR	43					No recovery, installed sand catcher
730	20		S05	42		Poorly graded SAND (SP); dense; trace SILT	1		GRAVEL in shoe 18" recovered
725	25		S06	37		Poorly graded SAND with SILT (SP-SM); dense; light brownish gray; dry; mostly fine and coarse SAND; few SILT; trace fine GRAVEL	1		PA: 10.2%<#200 sieve.
720	30								

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
720	30		S07	50/1"					
						End drilling at 32 feet due to refusal on cobbles Backfilled with cement-bentonite slurry			Refusal
715	35								
710	40								
705	45								
700	50								
695	55								
690	60								
685	65								

Report: GEO_10_SNA; File: BORINGS_B-1_B-10.GPJ; 11/13/2017 A-17-B4

Project: MSWD-Regional WWTP **Project Location: North Palm Springs, CA** **Project Number: 60551186 1.13**	# Log of Boring A-17-B5 Sheet 1 of 2

Date(s) Drilled	**9-28-17**	Logged By	**J. Leiva**	Checked By	**L. Vazquez**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" bullet bit**	Total Depth of Borehole	**40.3 feet**
Drill Rig Type	**Limited Acess Rig CME**	Drilling Contractor	**2R Drilling**	Surveyed Surface Elevation	**746 feet**
Water Level Depth (Feet)	**Not Encountered**	Sampling Method(s)	**Bulk, SPT (1.4" I.D.), Mod CAL**	Hammer Data	**Automatic Hammer, 140 lbs / 30" drop (Efficiency=81%)**
Borehole Backfill	**cement-bentonite slurry, covered with soil cuttings to match surface**	Location	**33.908070° N -116.530740° W**		

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	0					Poorly graded SAND with SILT (SP-SM); light brownish gray; dry; moslty fine, some coarse SAND; few SILT, trace fine GRAVEL	1		
745			S01						
	5		S02	9		Well graded SAND with SILT (SW-SM); loose; grades coarser	1		PA: 7.8%<#200 sieve, 12" recovered
740									
	10		S03	60		Poorly graded SAND with SILT (SP-SM); very dense; little fine to to coarse 2" GRAVEL	1		2" GRAVEL in shoe, bagged S-3-1
735									
	15		S04	24		medium dense			15" recovered
730									
	20		S05	26		light brownish gray	1		PA: 7.1%<#200 sieve, 18" recovered
725									
	25		S06	22		grayish brown	1		PA: 11.4%<#200 sieve. Non-Plastic, missing bottom 6" of sampler
720									
	30								

Report: GEO_10_SNA; File: BORINGS_B-1_B-10.GPJ; 11/13/2017 A-17-B5

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	30								
715			S07	29		SILTY SAND (SM); medium dense; light olive brown; some fine SAND; some fines Poorly graded SAND with SILT (SP-SM); medium dense; brownish yellow; dry; mostly fine and coarse SAND; few fines	3		PA: 49.3%<#200 sieve.
	35		S08	26			1		
710									
	40		S09	50/3"		grades fine (few SILT, moslty fine SAND)			
705						Total Depth = 40.25 feet Backfilled with cement-bentonite slurry			
	45								
700									
	50								
695									
	55								
690									
	60								
685									
	65								

Project: MSWD-Regional WWTP
Project Location: North Palm Springs, CA
Project Number: 60551186 1.13

Log of Boring A-17-B6

Date(s) Drilled	**9-29-17**	Logged By	**J. Leiva**	Checked By	**L. Vazquez**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" bullet bit**	Total Depth of Borehole	**50.1 feet**
Drill Rig Type	**Limited Acess Rig CME**	Drilling Contractor	**2R Drilling**	Surveyed Surface Elevation	**761 feet**
Water Level Depth (Feet)	**Not Encountered**	Sampling Method(s)	**Bulk, SPT (1.4" I.D.), Mod CAL**	Hammer Data	**Automatic Hammer, 140 lbs / 30" drop (Efficiency=81%)**
Borehole Backfill	**cement-bentonite slurry, covered with soil cuttings to match surface**	Location	**33.909800° N -116.530211° W**		

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
760	0		S01			Well graded SAND with SILT (SP-SM); light olive brown; dry; mostly fine, little coarse SAND; few SILT,	1		PA: 9.3%<#200 sieve
755	5		NR	25		Poorly graded SAND with SILT (SP-SM); medium dense			Driller dropped sample
750	10		S03	38		dense; becomes gray; few fine and coarse GRAVEL			fractured rock in barrel
745	15		S04	58		Poorly graded SAND (SP); very dense; light brownish gray; dry; mostly fine to coarse SAND; trace FINES	1		PA: 3.6%<#200 sieve, loose sand in sampler disturbed
740	20		S05	26		Poorly graded SAND with SILT (SP-SM); medium dense; brownish gray; dry; mostly fine to few coarse SAND; few SILT; trace fine GRAVEL			
735	25		S06	27		grayish brown; grades very fine; (mostly fine SAND; trace medium and coarse SAND few fines)	1		PA: 9.6%<#200 sieve
	30								

Elevation, feet	Depth, feet	Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
730	30		S07	40		dense; mostly fine and coarse SAND	1		2-inches of yellowish brown CLAY in the bottom of sampler and shoe
725	35		S08	50/6"		SILTY SAND (SM); very dense; yellowish brown; dry; mostly fine and medium SAND; some SILT			
720	40		S09	44			10		
						Poorly graded SAND with SILT and CLAY (SP-SM); dense; light olive brown; dry; moslty fine to coarse SAND; few fines	1		PA: 9.9%<#200 sieve
715	45		S10	64		very dense; grades coarse; no CLAY; trace fine GRAVEL			
710	50		S11	50/1"		few fine to coarse GRAVEL	1		
						Total Depth = 50.1 feet			
						Backfilled with cement-bentonite slurry			
705	55								
700	60								
	65								

Report: GEO_10_SNA; File: BORINGS_B-1_B-10.GPJ; 11/13/2017 A-17-B6

Project: MSWD-Regional WWTP
Project Location: North Palm Springs, CA
Project Number: 60551186 1.13

Log of Boring A-17-B7

Date(s) Drilled	9-29-17	Logged By	J. Leiva	Checked By	L. Vazquez
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8" bullet bit	Total Depth of Borehole	50.2 feet
Drill Rig Type	Limited Acess Rig CME	Drilling Contractor	2R Drilling	Surveyed Surface Elevation	760 feet
Water Level Depth (Feet)	Not Encountered	Sampling Method(s)	Bulk, SPT (1.4" I.D.), Mod CAL	Hammer Data	Automatic Hammer, 140 lbs / 30" drop (Efficiency=81%)
Borehole Backfill	cement-bentonite slurry, covered with soil cuttings to match surface	Location	33.909720° N -116.530800° W		

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
760	0					Poorly graded SAND with SILT (SP-SM); grayish brown; dry; mostly fine and medium SAND, little coarse SAND; few SILt; trace fine GRAVEL	1		WA: 9.4%<#200
			S01						
755	5		S02	22		medium dense			water leaked in the boring from the drill rig , fractured rock in barrel
750	10		S03	37		Well graded SAND (SW); light brownish gray; dense; grades coarse; few fine to coarse GRAVEL; trace SILT	1		PA: 4.6%<#200
745	15		S04	30		Poorly graded SAND (SP); little fine and coarse GRAVEL	1		
740	20		S05	21		Well graded SAND with SILT (SW-SM); medium dense; light brownish gray; grades coarse; mostly fine and little coarse SAND, few FINES; trace fine GRAVEL	1		PA: 8.5%<#200
735	25		S06	27		Poorly graded SAND with SILT (SP-SM); medium dense; mostly fine and little coarse SAND; few FINES; trace fine GRAVEL			
730	30								

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
730	30		S07	19			1		
						SILTY SAND (SM); Dry; very dense; reddish brown; dry; mostly fine and medium SAND; little SILT; trace fine and coarse GRAVEL			PP = 3.75
725	35		S08	55		dark yellowish brown	2		PA: 19.8%<#200, LL= 24 PI=1
720	40		S09	25		medium dense			
715	45		S10	49		SILT (ML): reddish brown; dense; dry			
						Poorly graded SAND with SILT (SP-SM); dense; brownish gray; dry; mostly fine and medium SAND, little coarse SAND; few SILT	1		fractured rock in barrel
710	50		S11	50/2"		very dense; damp			
						Total Depth = 50.2 feet			
						Backfilled with cement-bentonite slurry			
705	55								
700	60								
695	65								

Project: MSWD-Regional WWTP Project Location: North Palm Springs, CA Project Number: 60551186 1.13	**Log of Boring A-17-B8** Sheet 1 of 1

Date(s) Drilled	**9-26-17**	Logged By	**J. Leiva**	Checked By	**L. Vazquez**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" bullet bit**	Total Depth of Borehole	**20.9 feet**
Drill Rig Type	**Limited Acess Rig CME**	Drilling Contractor	**2R Drilling**	Approximate Surface Elevation	**1028 feet**
Water Level Depth (Feet)	**Not Encountered**	Sampling Method(s)	**Bulk, SPT (1.4" I.D.), Mod CAL**	Hammer Data	**Automatic Hammer, 140 lbs / 30" drop**
Borehole Backfill	**Temporary well installed for infiltration testing**	Location	**33.946298° N -116.535102° W**	Hammer Efficiency Rating (ERi)	**81** %

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Well constructed	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	0		S01			Well graded SAND with SILT (SW-SM); grayish brown; dry; mostly fine to medium SAND; few FINES; trace fine GRAVEL		1		PA: 9.9%<#200
1025										
	5		S02	46		Poorly graded SAND with SILT (SP-SM); dense; grayish brown; grades coarser SAND		1	112	
1020										
	10		S03	73		Well graded SAND with SILT (SW-SM); very dense; pale light brownish gray; dry; mostly fine to medium SAND; few FINES; trace fine GRAVEL		0		PA: 8.9%<#200, fractured rock
1015										
	15		S04	50/6"		Poorly graded SAND with SILT (SP-SM); grades coarser SAND				
1010										
	20		S05	50/5"						
1005						Total Depth = 20.9 feet Temporary monitoring well installed 9/26/17 Temporary monitoring well over drilled with 8" hollow stem auger on 10/2/17. PVC pipe and screen removed and backfilled with cement-bentonite slurry Well Notes: 0'-5' Backfill: cement-bentonite slurry 5'-6' Backfill: Pel-Plug Pellets 6'-20.9' Backfill: Backfill: #2/12 Clean graded sand 2" I.D Solid PVC pipe 0'-9' 2" I.D. Schedule-40 slotted PVC (size 0.010); 9.5'-20.5'				
	25									
1000										
	30									

Project: MSWD-Regional WWTP Project Location: North Palm Springs, CA Project Number: 60551186 1.13	**Log of Boring A-17-B9** Sheet 1 of 2

Date(s) Drilled	9-26-17	Logged By	J. Leiva	Checked By	L. Vazquez
Drilling Method	Hollow Stem Auger	Drill Bit Size/Type	8" bullet bit	Total Depth of Borehole	50.5 feet
Drill Rig Type	Limited Acess Rig CME	Drilling Contractor	2R Drilling	Surveyed Surface Elevation	996 feet
Water Level Depth (Feet)	Not Encountered	Sampling Method(s)	Bulk, SPT (1.4" I.D.), Mod CAL	Hammer Data	Automatic Hammer, 140 lbs / 30" drop (Efficiency=81%)
Borehole Backfill	cement-bentonite slurry, covered with soil cuttings to match surface	Location	33.942640° N -116.533340° W		

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	0					Poorly graded SAND with SILT (SP-SM); grayish brown; dry; moslty fine SAND; few FINES; trace fine GRAVEL	1		
995			S01						
	5		S02	16		medium dense; brownish gray	0		
990									
	10		S03	50/4"		very dense			
985									Refusal at 11' moved 5' SE for second attempt
	15		S04	28		SILTY SAND (SM); medium dense; grayish brown; mostly fine SAND; little SILT	1		
980									
	20		S05	70		Poorly graded SAND (SP); very dense; pale yellowish brown; mostly fine and coarse SAND; trace FINES; few fine GRAVEL	0		
975									
	25		S06	50/6"		1 1/2" layer of SILTY SAND (SM)			
970									
	30								

Project: MSWD-Regional WWTP
Project Location: North Palm Springs, CA
Project Number: 60551186 1.13

Log of Boring A-17-B9

Elevation, feet	Depth, feet	SAMPLES Type	Number	Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
965	30		S07	59		Well graded SAND with SILT (SW-SM); very dense; brownish gray; few SILT	1		
960	35		NR	74					
955	40		S09	49		SILTY SAND (SM); dense; pale yellowish brown; mostly fine SAND; little SILT	1		
950	45		S10	52		Well graded SAND with SILT (SW-SM); very dense; brownish gray; dry; mostly fine and coarse SAND; few fines; trace fine gravel	1		
945	50		NR	50/4"					
						Total Depth = 50.5 feet			
						Backfilled with cement-bentonite slurry			
940	55								
935	60								
	65								

Project: MSWD-Regional WWTP
Project Location: North Palm Springs, CA
Project Number: 60551186 1.13

Log of Boring A-17-B10

Date(s) Drilled	**9-26-17**	Logged By	**J. Leiva**	Checked By	**L. Vazquez**
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**8" bullet bit**	Total Depth of Borehole	**21.0 feet**
Drill Rig Type	**Limited Acess Rig CME**	Drilling Contractor	**2R Drilling**	Approximate Surface Elevation	**984 feet**
Water Level Depth (Feet)	**Not Encountered**	Sampling Method(s)	**Bulk, SPT (1.4" I.D.), Mod CAL**	Hammer Data	**Automatic Hammer, 140 lbs / 30" drop**
Borehole Backfill	**Temporary well installed for infiltration testing**	Location	**33.940944° N -116.533340°° W**	Hammer Efficiency Rating (ERi)	**81** %

Elevation, feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	MATERIAL DESCRIPTION	Well constructed	Water Content, %	Dry Density, pcf	REMARKS AND OTHER TESTS
	0	Bulk	S01			Poorly graded SAND with SILT (SP-SM); brownish gray; mostly fine to coarse SAND; few fines; trace organic material (roots and grass)		0		
980										
	5		S02	23		medium dense; light brownish		1	98	
975										
	10		S03	87/9"		SILTY SAND (SM); very dense; brownish gray; mostly fine SAND; little SILT		0		4" recovered coarse Gravel in barrel
970										
	15		S04	50/4"				0		Rig chatter
965										
	20		S05	50/6"		few fine to coarse GRAVEL				
960						Total Depth = 21 feet Temporary monitoring well installed 9/26/17 Temporary monitoring well over drilled with 8" hollow stem auger on 10/2/17. PVC pipe and screen removed and backfilled with cement-bentonite slurry. Well Notes: 0'-5' Backfill: Cement Grout 5'-6' Backfill: Pel-Plug Pellets 6'-21' Backfill: Backfill: #2/12 Clean graded sand 2" I.D Solid PVC pipe 0'-9.5' "2"" I.D. Schedule-40 slotted PVC (size 0.010); 9.5'-20.5'"				
	25									
955										
	30									

GeoLogic Associates
Boring Log

BORING NO.: B-1

PAGE: 1 OF 1

JOB NO.: 2004-155
SITE LOCATION: GARNET WELL, DESERT HOT SPRINGS, CA
DRILLING METHOD: 8 3/4" ø HOLLOW STEM AUGER
CONTRACTOR: C & K DRILLING
LOGGED BY: T. PRIMAS

DATE STARTED: 8/23/2004
DATE FINISHED: 8/23/2004
ELEVATION: 756 FEET MSL (RW BECK, 2004)

GW DEPTH: NOT ENCOUNTERED
CAVING: NONE OBSERVED
TOTAL DEPTH: 30.5 FEET

PID READING (PPM)	LABORATORY TESTING (SEE KEY)	DRY DENSITY (LBS/CU. FT.)	MOISTURE (%)	BLOWS (COUNT/FT.)	SAMPLE SIZE (INCHES)	SAMPLE NO.	DEPTH IN FEET	DEPTH IN METERS	MATERIAL SYMBOL	USCS/GEOLOGIC FORMATION	DESCRIPTION
					BULK	1	0	0		SW	ALLUVIUM: LIGHT BROWNISH GRAY (5YR 6/1), FINE TO COARSE SAND WITH SCATTERED GRAVEL.
				17	1.4	2		1			
					BULK	3					
				23	1.4	4	5				
				11	1.4	5		2			
				17	1.4	6	10	3			
								4			
				26	1.4	7	15	5			
								6			....AT 19 FEET: INCREASING SAND SIZE GRAVEL
				23	1.4	8	20				
								7			
				32	1.4	9	25	8			
								9			....AT 29 FEET: INCREASING COBBLES
				100+	1.4	10	30				

NOTES:

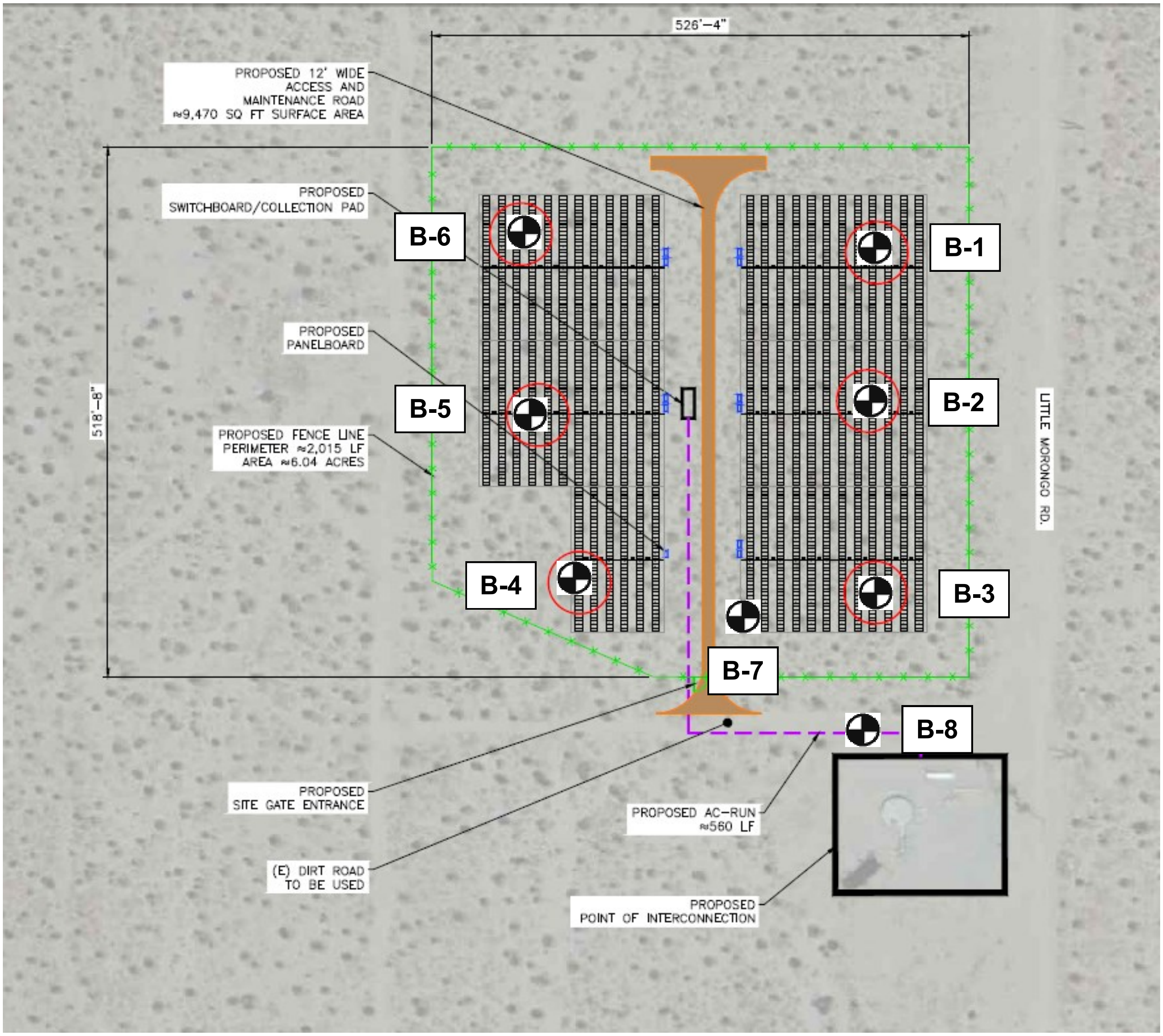
1. TOTAL BORING DEPTH = 30.5 FEET.
2. SAMPLER DRIVEN BY A 140-POUND HAMMER WITH A 30-INCH DROP.
3. NO GROUNDWATER OBSERVED AT TIME OF DRILLING.
4. BORING BACKFILLED WITH CUTTINGS ON 8/23/2004.
5. BORING CAVED UP TO 1 FOOT DEPTH AFTER AUGER WITHDRAWAL.

The data presented on this log is a simplification of actual conditions encountered and applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change with the passage of time.

LEGEND

B-1 Approximate Boring Location

BSK Job No. G15-068-11B
June 2015
Figure A-2



Map Reference: SunPower - Mission Springs WD Well 33

N

Approximate Scale

0 155 feet

1 inch = 155 feet

BORING LOCATION MAP

GEOTECHNICAL ENGINEERING INVESTIGATION
SUNPOWER MISSION SPRINGS
SOLAR PROJECT
NORTH PALM SPRINGS, CALIFORNIA

BSK Associates Engineers & Laboratories

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than Half >#200	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL- SAND MIXTURES
		GRAVELS WITH OVER 15% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 15% FINES	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half <#200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS , MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

Note: Dual symbols are used to indicate borderline soil classifications.

	Pushed Shelby Tube	RV	R-Value
	Standard Penetration Test	SA	Sieve Analysis
	Modified California	SW	Swell Test
	Auger Cuttings	TC	Cyclic Triaxial
	Grab Sample	TX	Unconsolidated Undrained Triaxial
	Sample Attempt with No Recovery	TV	Torvane Shear
CA	Chemical Analysis	UC	Unconfined Compression
CN	Consolidation	(1.2)	(Shear Strength, ksf)
CP	Compaction	WA	Wash Analysis
DS	Direct Shear	(20)	(with % Passing No. 200 Sieve)
PM	Permeability	∇	Water Level at Time of Drilling
PP	Pocket Penetrometer	▼	Water Level after Drilling (with date measured)

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA
Unified Soil Classification System

BSK Associates Engineers & Laboratories

PLATE: Figure A-4

LOG OF BORING NO. B-1

BSK Associates
700 22nd Street
Bakersfield, CA 93301

Project Name: **Mission Springs Solar Project**
Project Number: **G15 068 10B**
Project Location: **North Palm Springs, California**
Logged by: **C. Rozell**
Checked by: **A. Terronez**

Surface El.:
Location:

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
0		SP: SAND: Light Olive Brown; medium to coarse grained; dry; trace of fine grained sand.										
		... MEDIUM DENSE SAND: Light Olive Gray; medium to coarse grained; dry; trace of fine grained sand.			27				0			
5		... trace of gravel.			30				0			
10		... VERY DENSE SAND: Light Olive Gray; medium to coarse grained; dry; trace of fine grained sand.			50/ 6"				0			
15		... fine to coarse grained; dry; gravel and cobbles encountered.			50/ 6"				0			
		End of boring. Drilling refusal due to cobbles.										
20												

Completion Depth: 15.5
Date Started: 5/15/15
Date Completed: 5/15/15
California Sampler: 2.5 inch inner diameter
SPT Sampler: 1.4 inch inner diameter

Drilling Equipment: Mobile B-61 Drill Rig
Drilling Method: Hollow Stem Auger w/ Auto Trip Hammer
Drive Weight: 140 pounds
Hole Diameter: 8 inches
Drop: 30 inches
Remarks: Borings backfilled with soil cuttings. GW not encountered

LOG OF BORING NO. B-2

BSK Associates
700 22nd Street
Bakersfield, CA 93301

Project Name: **Mission Springs Solar Project**
Project Number: **G15 068 10B**
Project Location: **North Palm Springs, California**
Logged by: **C. Rozell**
Checked by: **A. Terronez**

Surface El.:
Location:

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		SP: SAND: Light Olive Brown; medium to coarse grained; dry; trace of fine grained sand.										
		... MEDIUM DENSE SAND: Light Olive Gray; fine to coarse grained; dry; cobbles encountered.			17				2			
5		... trace of gravel.			27				1			
10		" "			17			101	0			
		End of boring. Drilling refusal due to cobbles.										
15												
20												

Completion Depth: 12.0
Date Started: 5/15/15
Date Completed: 5/15/15
California Sampler: 2.5 inch inner diameter
SPT Sampler: 1.4 inch inner diameter

Drilling Equipment: Mobile B-61 Drill Rig
Drilling Method: Hollow Stem Auger w/ Auto Trip Hammer
Drive Weight: 140 pounds
Hole Diameter: 8 inches
Drop: 30 inches
Remarks: Borings backfilled with soil cuttings. GW not encountered

GEO_TARGET BORING LOGS.GPJ GEOTECHNICAL 08.GDT 5/27/15

LOG OF BORING NO. B-3

BSK Associates
700 22nd Street
Bakersfield, CA 93301

Project Name: **Mission Springs Solar Project**
Project Number: **G15 068 10B**
Project Location: **North Palm Springs, California**
Logged by: **C. Rozell**
Checked by: **A. Terronez**

Surface El.:
Location:

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetro-meter, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
0		SP: SAND: Light Olive Brown; medium to coarse grained; dry; trace of fine grained sand.										
2		... MEDIUM DENSE SAND: Light Olive Gray; fine to coarse grained; dry; cobbles encountered.			33				0			
5		... Light Olive Brown; fine to coarse grained; dry; cobbles encountered.			39				1			
10		... VERY DENSE SAND: Light Olive Gray; fine to coarse grained; dry, cobbles encountered.			83				2			
12		End of boring. Drilling refusal due to cobbles.										
15												
20												

Completion Depth:	12.0	**Drilling Equipment:**	Mobile B-61 Drill Rig
Date Started:	5/15/15	**Drilling Method:**	Hollow Stem Auger w/ Auto Trip Hammer
Date Completed:	5/15/15	**Drive Weight:**	140 pounds
California Sampler:	2.5 inch inner diameter	**Hole Diameter:**	8 inches
SPT Sampler:	1.4 inch inner diameter	**Drop:**	30 inches
		Remarks:	Borings backfilled with soil cuttings. GW not encountered

LOG OF BORING NO. B-4

BSK Associates
Engineers & Laboratories
BSK Associates
700 22nd Street
Bakersfield, CA 93301

Project Name: **Mission Springs Solar Project**
Project Number: **G15 068 10B**
Project Location: **North Palm Springs, California**
Logged by: **C. Rozell**
Checked by: **A. Terronez**

Surface El.:
Location:

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetro-meter, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
0		SP: SAND: Light Olive Brown; medium to coarse grained; dry; trace of fine grained sand.										
2		... MEDIUM DENSE SAND: Light Olive Gray; fine to medium grained; dry; trace of coarse grained sand.			13				1			
5		... Light Brown; fine to coarse grained; dry; cobbles encountered.			30				0			
10		... VERY DENSE SAND: Light Olive Gray; fine to coarse grained; dry; larger cobbles encountered.			64				0			
15		SP-SM: MEDIUM DENSE SAND TO SILTY SAND: Light Olive Gray; fine to coarse grained; dry; cobbles.			25				0			
16.5		End of boring.										
20												

Completion Depth: 16.5
Date Started: 5/15/15
Date Completed: 5/15/15
California Sampler: 2.5 inch inner diameter
SPT Sampler: 1.4 inch inner diameter

Drilling Equipment: Mobile B-61 Drill Rig
Drilling Method: Hollow Stem Auger w/ Auto Trip Hammer
Drive Weight: 140 pounds
Hole Diameter: 8 inches
Drop: 30 inches
Remarks: Borings backfilled with soil cuttings. GW not encountered

BSK Associates Engineers & Laboratories
BSK Associates
700 22nd Street
Bakersfield, CA 93301

LOG OF BORING NO. B-5

Project Name: **Mission Springs Solar Project**
Project Number: **G15 068 10B**
Project Location: **North Palm Springs, California**
Logged by: **C. Rozell**
Checked by: **A. Terronez**

Surface El.:
Location:

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		SP: SAND: Light Olive Brown; medium to coarse grained; dry; trace of fine grained sand.										
		... MEDIUM DENSE SAND: Light Brown; medium to coarse grained; dry; trace of fine grained sand, cobbles encountered.			36				1			
5		" "			39				1			
10		" "			43				0			
15		... fine to coarse grained; dry.			21				1			
		End of boring.										
20												

Completion Depth: 16.5
Date Started: 5/15/15
Date Completed: 5/15/15
California Sampler: 2.5 inch inner diameter
SPT Sampler: 1.4 inch inner diameter

Drilling Equipment: Mobile B-61 Drill Rig
Drilling Method: Hollow Stem Auger w/ Auto Trip Hammer
Drive Weight: 140 pounds
Hole Diameter: 8 inches
Drop: 30 inches
Remarks: Borings backfilled with soil cuttings. GW not encountered

LOG OF BORING NO. B-6

BSK Associates
Engineers & Laboratories

BSK Associates
700 22nd Street
Bakersfield, CA 93301

Project Name: **Mission Springs Solar Project**
Project Number: **G15 068 10B**
Project Location: **North Palm Springs, California**
Logged by: **C. Rozell**
Checked by: **A. Terronez**

Surface El.:
Location:

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
0	SP	SP: SAND: Light Olive Brown; medium to coarse grained; dry; trace of fine grained sand.										
2		... LOOSE SAND: Light Olive Gray; fine grained; dry; trace of medium grained sand.	California		14				1			
5		... MEDIUM DENSE SAND: Light Olive Gray; fine grained; dry; trace of medium grained sand, rock in tube.	California		20				0			
10		" "	California		28				0			
15		... Light Olive Brown; fine to coarse grained; dry; gravel and cobbles encountered.	SPT		42				0			
16.5		End of boring.										

Completion Depth: 16.5
Date Started: 5/15/15
Date Completed: 5/15/15
California Sampler: 2.5 inch inner diameter
SPT Sampler: 1.4 inch inner diameter

Drilling Equipment: Mobile B-61 Drill Rig
Drilling Method: Hollow Stem Auger w/ Auto Trip Hammer
Drive Weight: 140 pounds
Hole Diameter: 8 inches
Drop: 30 inches
Remarks: Borings backfilled with soil cuttings. GW not encountered

BSK Associates
Engineers & Laboratories

BSK Associates
700 22nd Street
Bakersfield, CA 93301

LOG OF BORING NO. B-7

Project Name: **Mission Springs Solar Project**
Project Number: **G15 068 10B**
Project Location: **North Palm Springs, California**
Logged by: **C. Rozell**
Checked by: **A. Terronez**

Surface El.:
Location:

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetrometer, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
0–5		SP: SAND: Light Olive Brown; medium to coarse grained; dry; trace of fine grained sand.										
5		End of boring.										
10												
15												
20												

Completion Depth: 5.0
Date Started: 5/15/15
Date Completed: 5/15/15
California Sampler: 2.5 inch inner diameter
SPT Sampler: 1.4 inch inner diameter

Drilling Equipment: Mobile B-61 Drill Rig
Drilling Method: Hollow Stem Auger w/ Auto Trip Hammer
Drive Weight: 140 pounds
Hole Diameter: 8 inches
Drop: 30 inches
Remarks: Borings backfilled with soil cuttings. GW not encountered

LOG OF BORING NO. B-8

BSK Associates
700 22nd Street
Bakersfield, CA 93301

Project Name:	**Mission Springs Solar Project**
Project Number:	**G15 068 10B**
Project Location:	**North Palm Springs, California**
Logged by:	**C. Rozell**
Checked by:	**A. Terronez**

Surface El.:
Location:

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Samples	Sample Number	Penetration Blows / Foot	Pocket Penetro-meter, TSF	% Passing No. 200 Sieve	In-Situ Dry Weight (pcf)	In-Situ Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index
		SP: SAND: Light Olive Brown; medium to coarse grained; dry; trace of fine grained sand.										
		End of boring.										
5												
10												
15												
20												

Completion Depth:	4.5	**Drilling Equipment:**	Mobile B-61 Drill Rig
Date Started:	5/15/15	**Drilling Method:**	Hollow Stem Auger w/ Auto Trip Hammer
Date Completed:	5/15/15	**Drive Weight:**	140 pounds
California Sampler:	2.5 inch inner diameter	**Hole Diameter:**	8 inches
SPT Sampler:	1.4 inch inner diameter	**Drop:**	30 inches
		Remarks:	Borings backfilled with soil cuttings. GW not encountered

Appendix B

Infiltration Test Results

AECOM

Boring/Excavation Percolation Testing Field Log

Date 9/29/2017

Project Location	MSWD Mission Springs	**Boring/Test Number**	B-1
Earth Description	(SP-SM)	**Diameter of Boring (in)**	8
		Diameter of Casing (in)	2
Tested by	Luis Vazquez	**Depth of Boring (ft)**	20.35
Liquid Description	Water	**Depth to Invert of BMP (in)**	120
Measurement Method	Water Level Meter	**Depth to Initial Water Depth (in) (d_1)**	224
		Depth to Water Table	N/A

Time Interval Standard

Pre-Soak Period	11:55 AM - 1:55 PM	**Water Remaining In Boring (Y/N)**	N
Standard Period	1:55 PM - 3:00 PM	**Standard Time Interval Between Readings**	10 min

Reading Number	Time Start / End (hh:mm)	Elasped Time Δtime (mins)	Depth to Water: Initial / Final (in.)	Water Drop During Standard Time Interval Δd (in)	Percolation Rate for Reading (min/in)	Infiltration Rate (in/hr)	Soil Description/Notes/Comments
Trial 1				0.0			Water drained out very quickly
Trial 2				0.0			Water drained out very quickly
1	1:55 PM / 2:05 PM	10	224.4 / 244.2	19.8	0.51	2.04	The water drained out in less than10 min
2	2:05 PM / 2:10 PM	5	235.2 / 239.2	4.0	1.26	0.80	
3	2:15 PM / 2:20 PM	5	234.0 / 238.6	4.6	1.10	0.93	
4	2:25 PM / 2:30 PM	5	214.2 / 239.0	24.8	0.20	5.49	
5	2:35 PM / 2:40 PM	5	237.6 / 238.8	1.2	4.17	0.24	
6	2:45 PM / 2:50 PM	5	238.1 / 238.8	0.7	6.94	0.14	
7	2:55 PM / 3:00 PM	5	213.8 / 237.8	24.0	0.21	5.34	
8							
9							
10							
11							
12							
13							

d_1 = Initial water depth (in.)
Δd = Water drop of final period (in.)
DIA = Diameter of boring (in.)
r = Radius of boring (in.)
H_{avg} = Average head height over the time interval (in.)
ΔH = Change in height over the time interval (in.)
Δt = time interval (min.)

$$I_t = \frac{\Delta H \, 60 \, r}{\Delta t \, (r + 2 \, H_{avg})}$$

Tested Infiltration Rate = 5.34 in/hr

AECOM

Boring/Excavation Percolation Testing Field Log

Date 9/29/2017

Project Location	MSWD Mission Springs	**Boring/Test Number**	B-2
Earth Description	(SP-SM)	**Diameter of Boring (in)**	8 — **Diameter of Casing (in)** 2
Tested by	Luis Vazquez	**Depth of Boring (ft)**	20.35
Liquid Description	Water	**Depth to Invert of BMP (in)**	120
Measurement Method	Water Level Meter	**Depth to Initial Water Depth (in) (d_1)**	195
		Depth to Water Table	N/A

Time Interval Standard

Pre-Soak Period	8:00 AM - 10:00 AM	**Water Remaining In Boring (Y/N)**	N
Standard Period	10:00 AM - 11:40 AM	**Standard Time Interval Between Readings**	10 min

Reading Number	Time Start / End (hh:mm)	Elapsed Time Δtime (mins)	Depth to Water: Initial / Final (in.)	Water Drop During Standard Time Interval Δd (in)	Percolation Rate for Reading (min/in)	Infiltration Rate (in/hr)	Soil Description/Notes/Comments
Trial 1	10:00 AM / 10:25 AM	25	195.0 / 244.2	49.2	1.97	2.32	Water drained out very quickly
Trial 2				0.0			Water drained out very quickly
1	10:45 AM / 10:50 AM	5	201.6 / 234.6	33.0	0.15	7.91	
2	10:55 AM / 11:00 AM	5	198.6 / 234.2	35.6	0.14	8.69	
3	11:05 AM / 11:10 AM	5	199.4 / 231.8	32.4	0.15	7.96	
4	11:15 AM / 11:20 AM	5	201.4 / 239.0	37.7	0.13	8.85	
5	11:25 AM / 11:30 AM	5	201.1 / 232.1	31.0	0.16	7.54	
6	11:35 AM / 11:40 AM	5	195.8 / 231.8	36.0	0.14	9.02	
7							
8							
9							
10							
11							
12							
13							

d_1 = Initial water depth (in.)
Δd = Water drop of final period (in.)
DIA = Diameter of boring (in.)
r = Radius of boring (in.)
H_{avg} = Average head height over the time interval (in.)
ΔH = Change in height over the time interval (in.)
Δt = time interval (min.)

$$I_t = \frac{\Delta H \; 60 \; r}{\Delta t \; (r + 2 \; H_{avg})}$$

Tested Infiltration Rate = 9.02 in/hr

AECOM

Boring/Excavation Percolation Testing Field Log

Date 9/28/2017

Project Location	MSWD Mission Springs	**Boring/Test Number**	B-8		
Earth Description	(SP-SM)	**Diameter of Boring (in)**	10	**Diameter of Casing (in)**	2
Tested by	Luis Vazquez	**Depth of Boring (ft)**	20.33		
Liquid Description	Water	**Depth to Invert of BMP (in)**	120		
Measurement Method	Water Level Meter	**Depth to Initial Water Depth (in) (d_1)**	193.44		
		Depth to Water Table	N/A		

Time Interval Standard

Pre-Soak Period	3:00 PM - 5:00 PM	**Water Remaining In Boring (Y/N)**	N
Standard Period	10:00 AM - 5:48 PM	**Standard Time Interval Between Readings**	4 min

Reading Number	Time Start / End (hh:mm)	Elasped Time Δtime (mins)	Depth to Water: Initial / Final (in.)	Water Drop During Standard Time Interval Δd (in)	Percolation Rate for Reading (min/in)	Infiltration Rate (in/hr)	Soil Description/Notes/Comments
Trial 1							Water drained out very quickly
Trial 2							Water drained out very quickly
1	5:00 PM / 5:10 PM	10	193.4 / 231.0	37.6	0.27	5.95	Time constrains caused us to take faster readings
2	5:10 PM / 5:20 PM	10	231.0 / 244.0	13.0	0.77	1.62	
3	5:27 PM / 5:31 PM	4	177.6 / 211.2	33.6	0.12	16.38	
4	5:31 PM / 5:35 PM	4	211.2 / 226.3	15.1	0.26	5.60	

d_1 = Initial water depth (in.)
Δd = Water drop of final period (in.)
DIA = Diameter of boring (in.)
r = Radius of boring (in.)
H_{avg} = Average head height over the time interval (in.)
ΔH = Change in height over the time interval (in.)
Δt = time interval (min.)

$$I_t = \frac{\Delta H\ 60\ r}{\Delta t\ (r + 2\ H_{avg})}$$

Tested Infiltration Rate = 5.6 in/hr

AECOM

Boring/Excavation Percolation Testing Field Log

Date 9/29/2017

Project Location	MSWD Mission Springs	**Boring/Test Number**	B-10		
Earth Description	(SP-SM)	**Diameter of Boring (in)**	8	**Diameter of Casing (in)**	2
Tested by	Luis Vazquez	**Depth of Boring (ft)**	20.32		
Liquid Description	Water	**Depth to Invert of BMP (in)**	120		
Measurement Method	Water Level Meter	**Depth to Initial Water Depth (in) (d_1)**	152.8		
		Depth to Water Table	N/A		

Time Interval Standard

Pre-Soak Period	8:30 AM - 10:30 AM	**Water Remaining In Boring (Y/N)**	N
Standard Period	10:30 AM - 1:30 PM	**Standard Time Interval Between Readings**	10 min

Reading Number	Time Start / End (hh:mm)	Elapsed Time Δtime (mins)	Depth to Water: Initial / Final (in.)	Water Drop During Standard Time Interval Δd (in)	Percolation Rate for Reading (min/in)	Infiltration Rate (in/hr)	Soil Description/Notes/Comments
Trial 1	10:30 AM / 10:55 AM	25	152.8 / 233.4	80.6	0.31	5.16	Water drained out very quickly
Trial 2	11:15 AM / 11:40 AM	25	151.2 / 233.0	81.8	0.31	5.30	Water drained out very quickly
1	12:00 PM / 12:10 PM	10	147.6 / 206.5	58.9	0.17	11.97	
2	12:10 PM / 12:20 PM	10	206.5 / 226.8	20.3	0.49	2.47	
3	12:20 PM / 12:30 PM	10	226.8 / 232.8	6.0	1.67	0.64	
4	12:30 PM / 12:40 PM	10	232.8 / 236.6	3.8	2.60	0.39	
5	12:40 PM / 12:50 PM	10	236.6 / 238.9	2.3	4.39	0.23	
6	1:10 PM / 1:20 PM	10	159.6 / 230.4	70.8	0.14	11.03	
7							
8							
9							
10							
11							
12							
13							

d_1 = Initial water depth (in.)
Δd = Water drop of final period (in.)
DIA = Diameter of boring (in.)
r = Radius of boring (in.)
H_{avg} = Average head height over the time interval (in.)
ΔH = Change in height over the time interval (in.)
Δt = time interval (min.)

$$I_t = \frac{\Delta H \; 60 \; r}{\Delta t \; (r + 2 \; H_{avg})}$$

Tested Infiltration Rate = 11.03 in/hr

Appendix C

Geotechnical Laboratory Results

Geotechnical soil samples obtained from the borings were carefully sealed and packaged in the field to reduce moisture loss and disturbance. The samples were subsequently delivered to our laboratory where they were further examined and classified. Selected representative samples were tested to evaluate water content, in-situ dry density, fines content, Atterberg limits, shear strength, corrosivity, swelling potential, and R- value. All tests discussed below were performed in accordance with the latest American Society of Testing and Materials (ASTM), or California Test Method (CTM) standards.

Water Content (ASTM D2216)

Water content tests were performed on selected soil/rock samples in general accordance with ASTM D2216, *Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*. The results of the tests are presented in Table C-1 and also presented on boring logs.

Soil Classification (ASTM D2488)

Soil identification and classification was performed on all soil samples obtained from the borings. The soil identification is based on visual examination and manual tests, in accordance with ASTM D2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedures).*

Moisture Content and Dry Density (ASTM D7263)

The density tests were performed on selected soil samples obtained from the borings. The dry density tests were performed in accordance with ASTM Test Methods D7263, *Standard Test Methods for Laboratory Determination of Density (Unit Weight) of Soil Specimens*. A summary of the results are presented on the Log of Borings in Appendix A as well as summarized in Table C-1.

Atterberg Limits (ASTM D4318)

Atterberg Limits test was performed to aid in classification and to evaluate the plasticity characteristics of fine-grained materials encountered in the borings. The test was performed in accordance with ASTM Test Method D4318, *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils*. The results of this test are presented on the Logs of Borings. Summary plots are plotted as Plasticity Charts (Figures C-1 and C-2).

Wash Analysis (ASTM D1140)

Percent passing no. 200 sieve tests were performed on selected soils samples obtained from the borings. These tests were performed to aid in classification of the soils and to help in evaluating the liquefaction potential of the soils. The tests were performed in accordance with ASTM Test Method D1140, *Standard Test Methods for Determining the Amount of Material Finer than 75-µm (No. 200) Sieve in Soils by Washing*. The results of the tests are presented in Table C-1 as well as shown on the Log of Borings in Appendix A.

Sieve Analysis (ASTM D6913)

Tests were performed to determine the particle size distribution of selected soil samples. These tests were performed in accordance with ASTM Test Method D6913, *Standard Test Methods for Particle-Size*

Distribution (Gradation) of Soils Using Sieve Analysis. Test results are appended as Particle Size Distribution Curves and presented within this Appendix C (Figures C-3 through C-23).

<u>Direct Shear Test (ASTM D3080)</u>

Consolidated-drained (saturated) direct shear tests were performed on relative undisturbed samples to evaluate shear strength parameters of the on-site soils. The direct shear tests were performed in accordance with ASTM Test Method D3080, *Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions*. The results of the direct shear tests are presented in Appendix C (Figures C-24 and C-25).

<u>Corrosivity Tests (CTM 417, 422 & 643)</u>

Selected representative samples obtained from the boring were tested for corrosion. Determination of the soluble sulfate and water-soluble chloride content of on-site soils and minimum resistivity and pH testing were conducted in accordance with CTM Test Methods: CTM 417, *Method of Testing Soils and Waters for Sulfate Content*; CTM 422, *Method of Testing Soils and Waters for Chloride Content*; and CTM 643, *Method for Determining Field and Laboratory Resistivity and pH Measurements for Soil and Water.* The results of the corrosion tests are presented in Appendix C (Figure C-26).

<u>One-Dimensional Swell/Collapse Potential (ASTM D4546)</u>

Selected samples were tested to determine the magnitude of swell or settlement of relatively undisturbed or compacted cohesive soil. Test methods were performed in accordance with ASTM D4546, *Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils*. The results are presented in Appendix C (Figure C-27).

<u>R-Value (CTM 301)</u>

Selected representative bulk samples obtained from the boring were tested to measure the response of a compacted sample of soil to a vertically applied pressure under specific conditions. The sand equivalent tests were performed in accordance with CTM 301, *Method for Determining the Resistance "R" Value of Treated and Untreated Bases, Subbases, and Basement Soils by the Stabilometer*. The results of the R-Value tests are presented in Appendix C (Figure C-28).

Table C-1: Santa Ana Geotechnical Laboratory Testing Summary

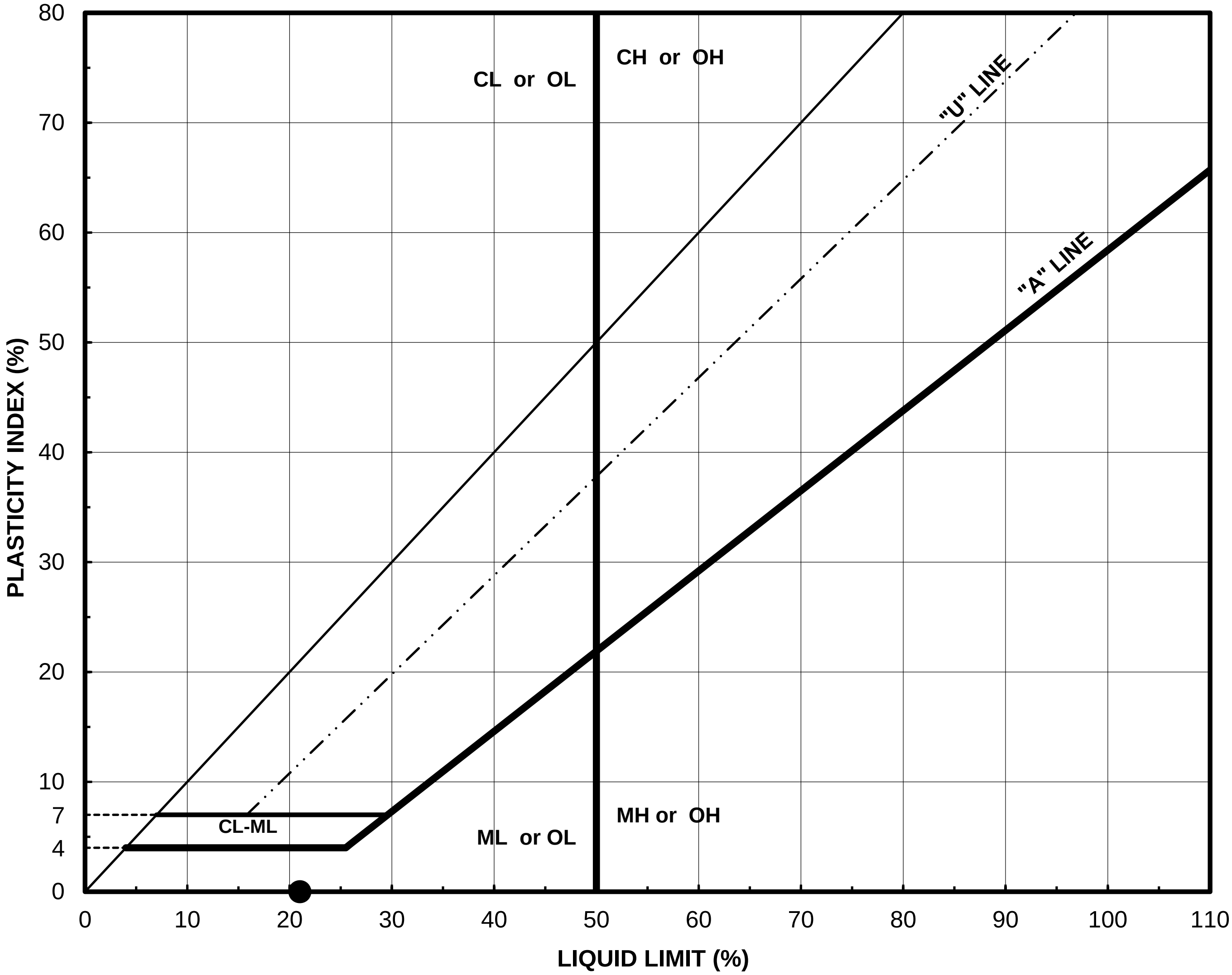
Project Name: MSWD Mission Springs
Project Number: 60551186
Project Engineer: MGS

Location				Initial Condition			Limits			Gradation			Direct Shear			Corrosivity Tests			
Boring Number	Sample Number	Depth (ft)	USCS Symbol	Water Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Liquid Limit	Plasticity Index	Liquidity Index	Gravel (%)	Sand (%)	Fines (%)	Normal Stress Sequence (ksf)	Peak Friction Angle (deg)	Strength Intercept (ksf)	Resistivity, ohm-cm	pH	Sulfate Content, ppm	Chloride Content, ppm
B-1	2	5.0	SP-SM	0.4						31.0	59.7	9.3							
B-1	3	10.0	SP-SM	0.5															
B-1	4	15.0	SW-SM	0.7						6.8	85.5	7.7							
B-1	5	20.0	SP-SM	22.1															
B-2	1	0-5	SP	1.1															
B-2	2	5.0	SP	1.4	112.3	110.7													
B-2	3	10.0	SW-SM	0.9						14.7	79.6	5.7							
B-2	4	15.0	SP-SM	0.8								8.6							
B-3	1	0-5	SP	0.4															
B-3	2	5.0	SP-SM	0.6	117.1	116.4				17.0	78.4	4.6	1,2,4	41	0				
B-3	3	10.0	SP-SM													9,600	9.8	17	2
B-3	5	20.0	SW-SM	0.7						3.2	90.7	6.1							
B-3	6B	30.3	SM	1.6			21	0	N/A			22.1							
B-3	7	35.0	SP	0.6															
B-4	2	5.0	SW-SM	0.8	119.2	118.2				2.0	90.3	7.7							
B-4	3	10.0	SP-SM	0.6								9.9							
B-4	4	20.0	SP	0.6															
B-4	5	25.0	SP-SM	0.7						11.8	78.0	10.2							
B-5	1	0-5	SM	0.5															
B-5	2	5.0	SW-SM	0.5						9.7	92.5	7.8							
B-5	3	10.0	SM/GP	0.5															
B-5	5	20.0	SP-SM	0.7								7.1							
B-5	6	25.0	SP-SM	0.9			Non-Plastic					11.4							
B-5	7B	30.5	SM	2.9								49.3							
B-5	8	35.0	SP-SM	0.7															
B-6	1	0-5	SW-SM	0.8						1.5	89.2	9.3							

Table C-1: Santa Ana Geotechnical Laboratory Testing Summary

Project Name: MSWD Mission Springs
Project Number: 60551186
Project Engineer: MGS

Location				Initial Condition			Limits			Gradation			Direct Shear			Corrosivity Tests			
Boring Number	Sample Number	Depth (ft)	USCS Symbol	Water Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Liquid Limit	Plasticity Index	Liquidity Index	Gravel (%)	Sand (%)	Fines (%)	Normal Stress Sequence (ksf)	Peak Friction Angle (deg)	Strength Intercept (ksf)	Resistivity, ohm-cm	pH	Sulfate Content, ppm	Chloride Content, ppm
B-6	3	15.0	SP	0.7						15.4	81.0	3.6							
B-6	5	25.0	SP-SM	0.7						2.5	87.9	9.6							
B-6	6	30.0	SP-SM	1.0															
B-6	8A	40.0	SM	9.6															
B-6	8B	41.0	SW-SM	0.8						20.8	69.3	9.9							
B-6	10	50.0	SP-SM	0.6												2,400	11.4	139	4.5
B-7	1	0-5	SP-SM	0.5								9.4							
B-7	3	10.0	SW	0.6						8.6	86.8	4.6							
B-7	4	15.0	SP	0.7															
B-7	5	20.0	SW-SM	0.8						7.7	83.8	8.5							
B-7	7A	30.0	SM	1.3															
B-7	8	35.0	SM	2.0			24	1	-21.03			19.8							
B-7	10B	45.5	SP-SM	0.7															
B-8	1	0-5	SW-SM	0.6						3.0	87.1	9.9							
B-8	2	5.0	SP-SM	1.0	113.5	112.4													
B-8	3	10.0	SW-SM	0.3						32.3	58.8	8.9							
B-9	1	0-5	SP-SM	0.6								7.5							
B-9	2	5.0	SP-SM	0.3						20.3	70.3	9.4							
B-9	4	15.0	SM	0.8								21.3							
B-9	5	20.0	SM	0.4															
B-9	7	30.0	SW-SM	0.5						23.2	70.5	6.3							
B-9	8	40.0	SM	0.8															
B-9	9	45.0	SW-SM	0.6						8.1	85.9	6.0							
B-10	1	0-5	SP-SM	0.4						2.8	91.4	5.8							
B-10	2	5.0	SP-SM	1.1	99.4	98.3													
B-10	3	10.0	SM	0.3						19.4	64.2	16.4							
B-10	4	15.0	SM	0.2															

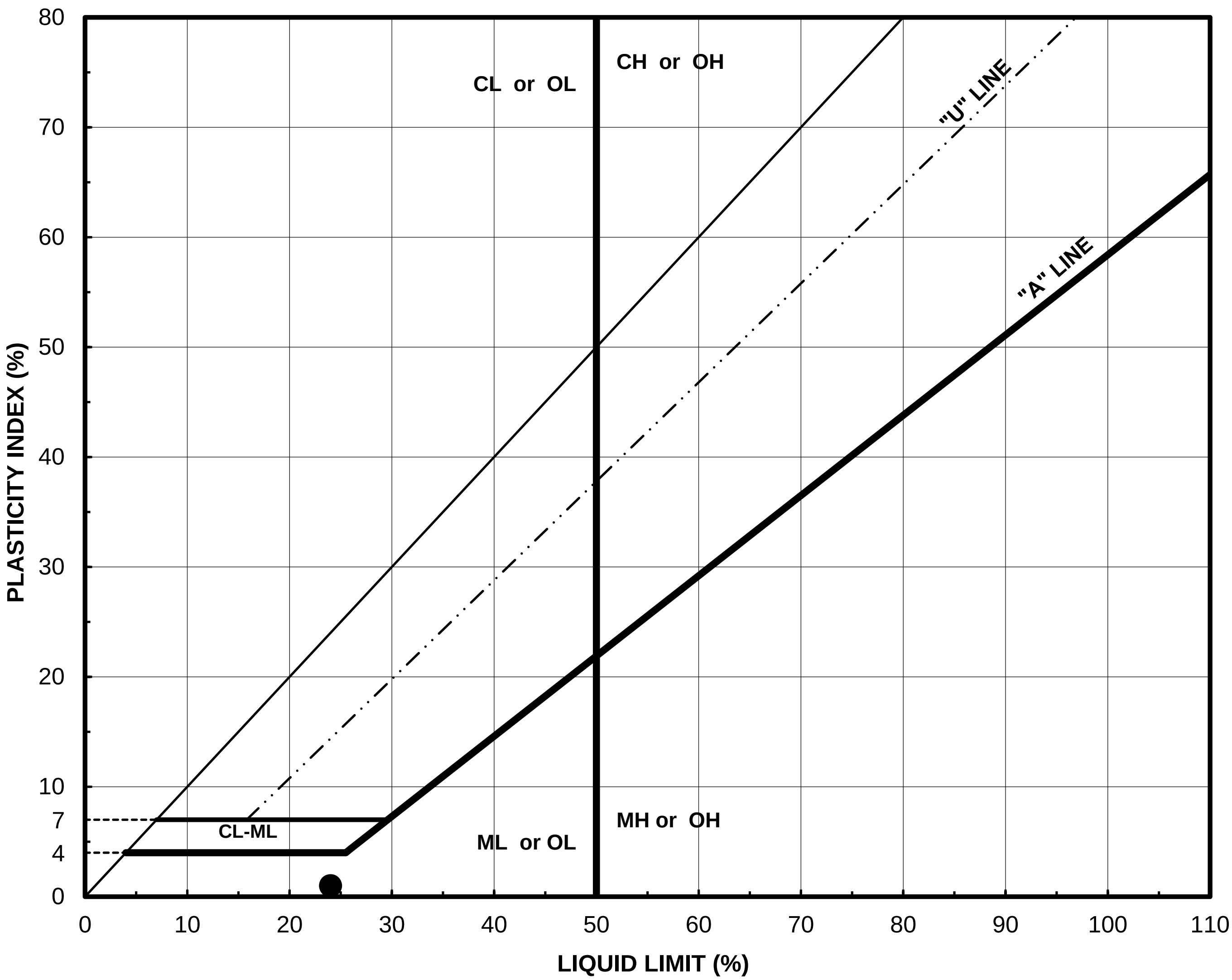


Boring Number	Sample Number	Depth (ft)	Water Content (%)	LL	PI	DESCRIPTION / CLASSIFICATION
B-3	6B	30.3	1.6	21	0	Light olive brown Silty SAND (SM)

Project Name: **MSWD Mission Springs** **PLASTICITY CHART**

Project Number: **60551186**

Figure C-1

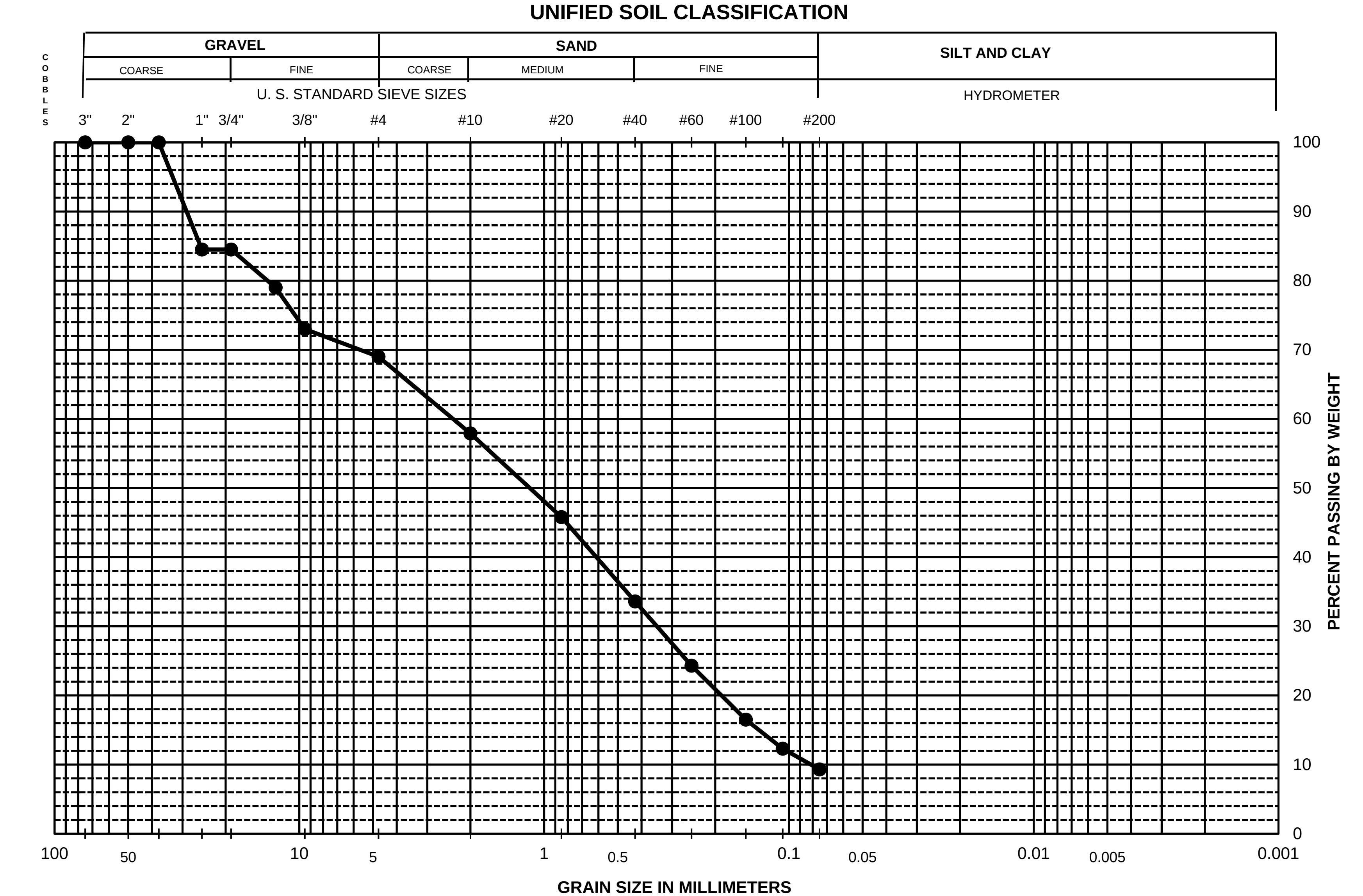


Boring Number	Sample Number	Depth (ft)	Water Content (%)	LL	PI	DESCRIPTION / CLASSIFICATION
B-7	8	35.0	2.0	24	1	Dark yellowish brown Silty SAND (SM)

Project Name: **MSWD Mission Springs** **PLASTICITY CHART**

Project Number: **60551186**

Figure C-2



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	84.5
3/4"	19.00	84.5
1/2"	12.50	79.0
3/8"	9.50	73.0
#4	4.75	69.0
#10	2.00	57.9
#20	0.850	45.8
#40	0.425	33.6
#60	0.250	24.3
#100	0.150	16.5
#140	0.106	12.3
#200	0.075	9.3
Hydrometer Analysis		

% Cobbles	---
% Gravel	31.0
% Sand	59.7
% Fines	9.3
D_{85}	25.000
D_{60}	2.356
D_{50}	1.144
D_{30}	0.346
D_{15}	0.133
D_{10}	0.081
C_u	29.0
C_c	0.6

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-1	2	5.0	●	0.4	---	---	---	**Light brownish gray Poorly Graded SAND with Silt and Gravel (SP-SM)**

PROJECT NAME: **MSWD Mission Springs**

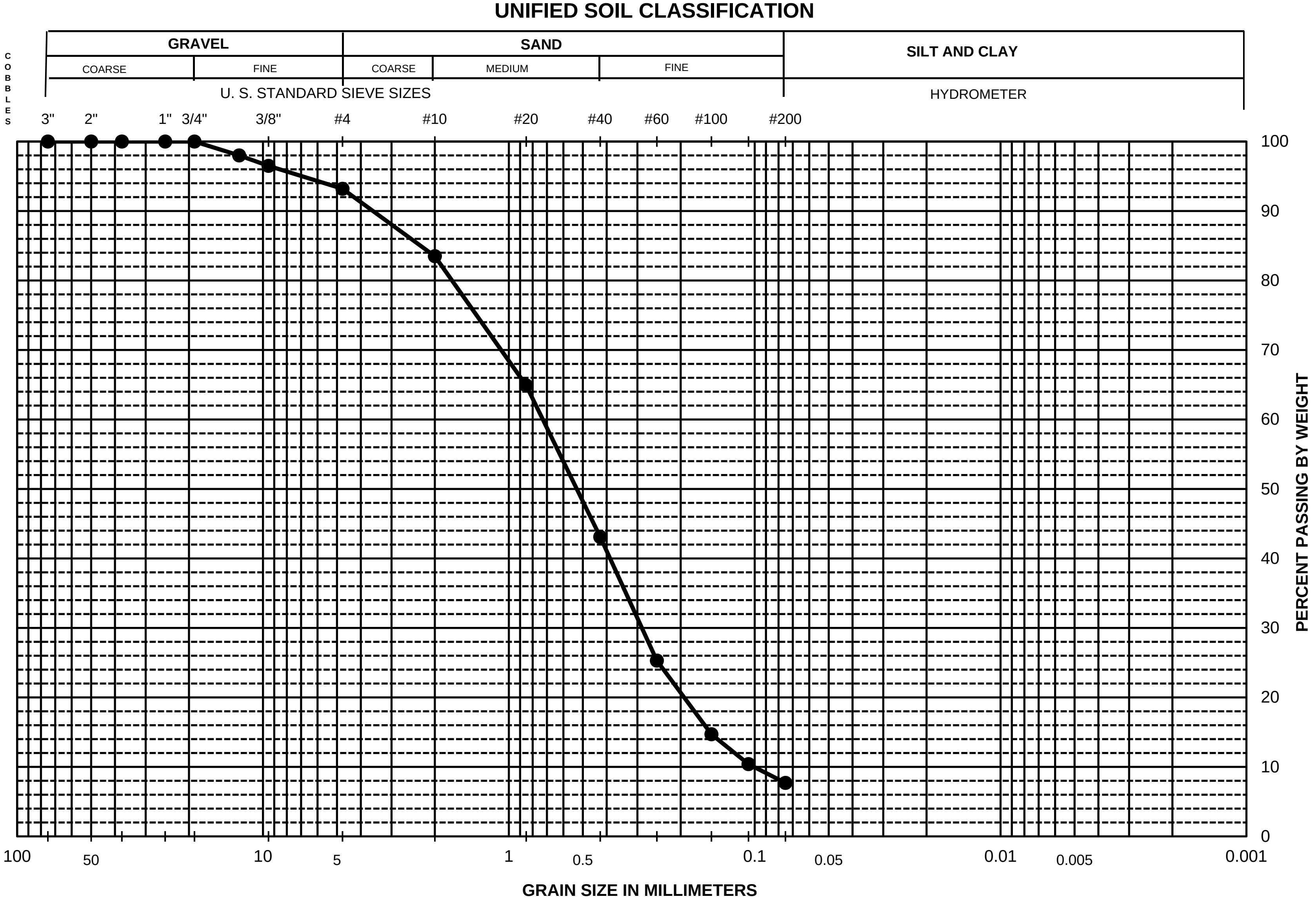
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-3

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-1\Sieve MSWD Mission Springs B-1 5 ft.xlsx

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	98.0
3/8"	9.50	96.5
#4	4.75	93.2
#10	2.00	83.5
#20	0.850	64.9
#40	0.425	43.1
#60	0.250	25.3
#100	0.150	14.7
#140	0.106	10.4
#200	0.075	7.7
Hydrometer Analysis		

% Cobbles	---
% Gravel	6.8
% Sand	85.5
% Fines	7.7
D_{85}	2.286
D_{60}	0.727
D_{50}	0.529
D_{30}	0.288
D_{15}	0.152
D_{10}	0.101
C_u	7.2
C_c	1.1

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-1	4	15.0	●	0.7	---	---	---	**Light brownish gray Well Graded SAND with Silt (SW-SM)**

PROJECT NAME: **MSWD Mission Springs**

PROJECT NUMBER: **60551186**

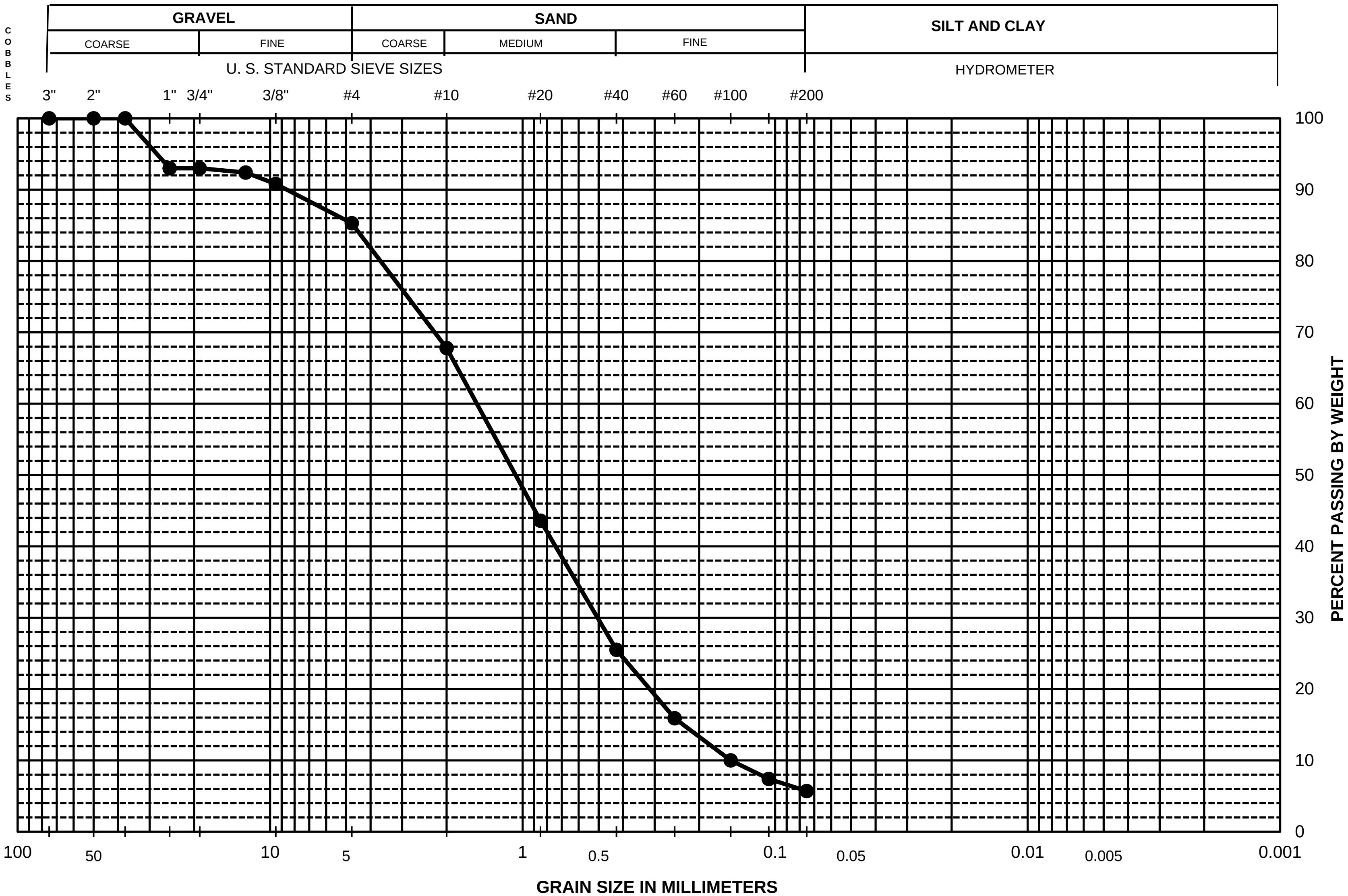
PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-4

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-1\Sieve MSWD Mission Springs B-1 15 ft.xlsx

AECOM

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	93.0
3/4"	19.00	93.0
1/2"	12.50	92.4
3/8"	9.50	90.8
#4	4.75	85.3
#10	2.00	67.8
#20	0.850	43.6
#40	0.425	25.5
#60	0.250	15.9
#100	0.150	10.0
#140	0.106	7.4
#200	0.075	5.7

Hydrometer Analysis

% Cobbles	---
% Gravel	14.7
% Sand	79.6
% Fines	5.7
D_{85}	4.680
D_{60}	1.518
D_{50}	1.066
D_{30}	0.505
D_{15}	0.231
D_{10}	0.150
C_u	10.1
C_c	1.1

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-2	3	10.0	●	0.9	---	---	---	**Light brownish gray Well Graded SAND with Silt (SW-SM)**

PROJECT NAME: **MSWD Mission Springs**

PROJECT NUMBER: **60551186**

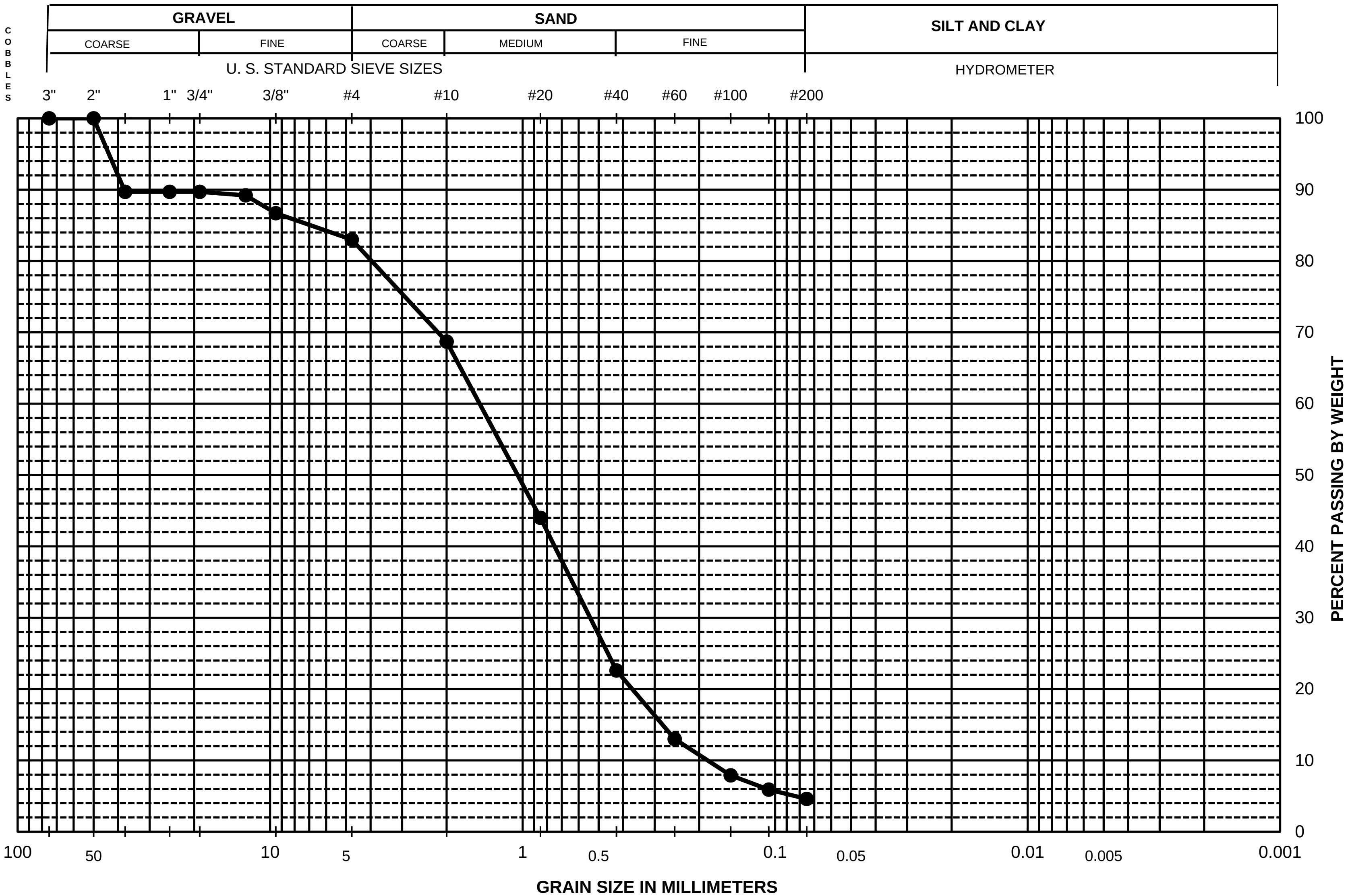
PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-5

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-2\Sieve MSWD Mission Springs B-2 10 ft.xlsx

AECOM

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	89.7
1"	25.0	89.7
3/4"	19.00	89.7
1/2"	12.50	89.2
3/8"	9.50	86.7
#4	4.75	83.0
#10	2.00	68.7
#20	0.850	44.0
#40	0.425	22.6
#60	0.250	13.0
#100	0.150	7.9
#140	0.106	5.9
#200	0.075	4.6

Hydrometer Analysis

% Cobbles	---
% Gravel	17.0
% Sand	78.4
% Fines	4.6
D_{85}	6.909
D_{60}	1.480
D_{50}	1.046
D_{30}	0.540
D_{15}	0.279
D_{10}	0.185
C_u	8.0
C_c	1.1

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-3	2	5.0	●	0.6	---	---	---	**Light brownish gray Well Graded SAND with Gravel (SW)**

PROJECT NAME: **MSWD Mission Springs**

PROJECT NUMBER: **60551186**

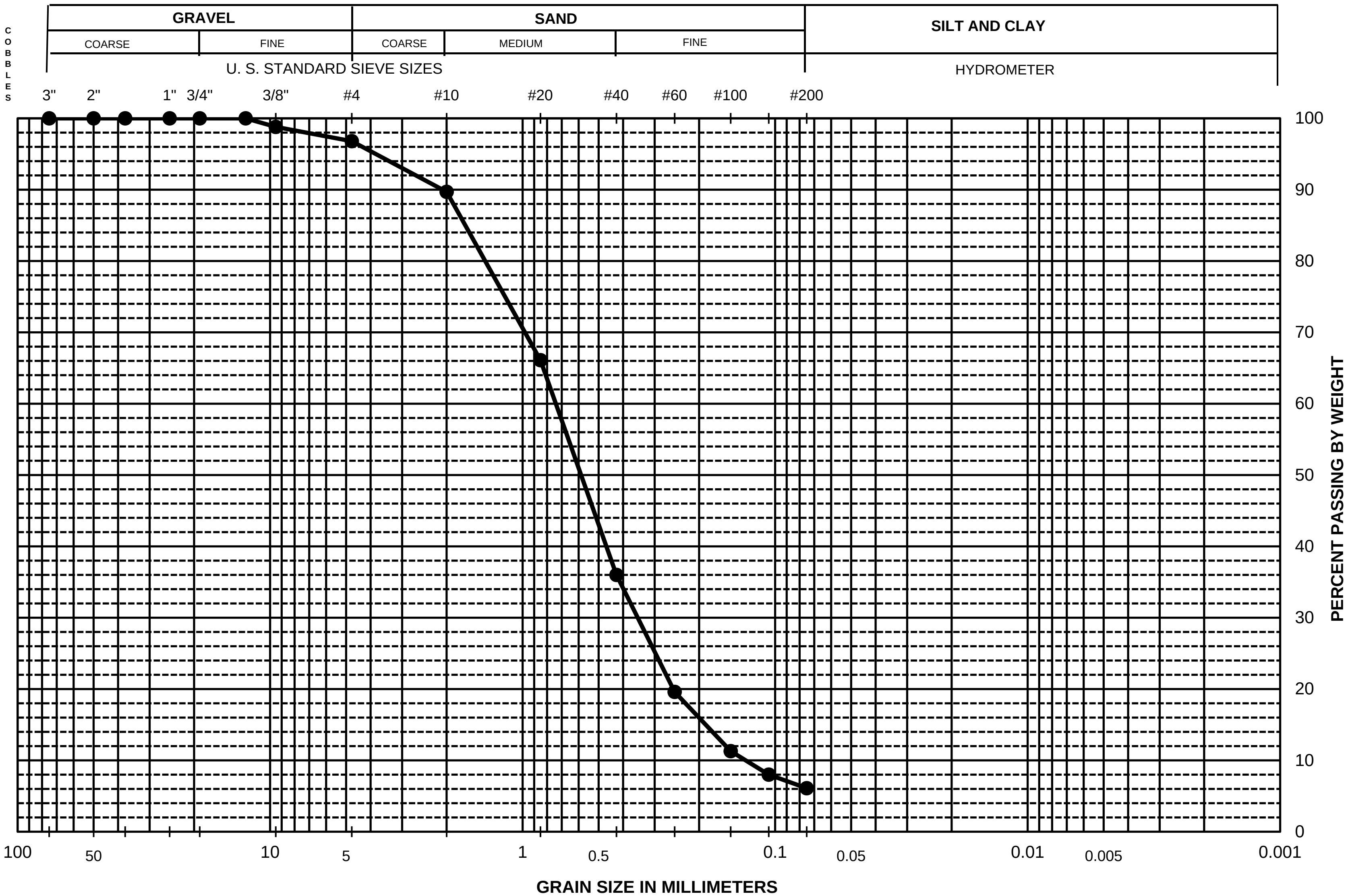
PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-6

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-3\Sieve MSWD Mission Springs B-3 5 ft.xlsx

AECOM

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	98.8
#4	4.75	96.8
#10	2.00	89.7
#20	0.850	66.1
#40	0.425	36.0
#60	0.250	19.6
#100	0.150	11.3
#140	0.106	8.0
#200	0.075	6.1
Hydrometer Analysis		

% Cobbles	---
% Gravel	3.2
% Sand	90.7
% Fines	6.1
D_{85}	1.687
D_{60}	0.739
D_{50}	0.587
D_{30}	0.350
D_{15}	0.188
D_{10}	0.131
C_u	5.6
C_c	1.3

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-3	5	20.0	●	0.7	---	---	---	**Light brownish gray Well Graded SAND with Silt (SW-SM)**

PROJECT NAME: **MSWD Mission Springs**

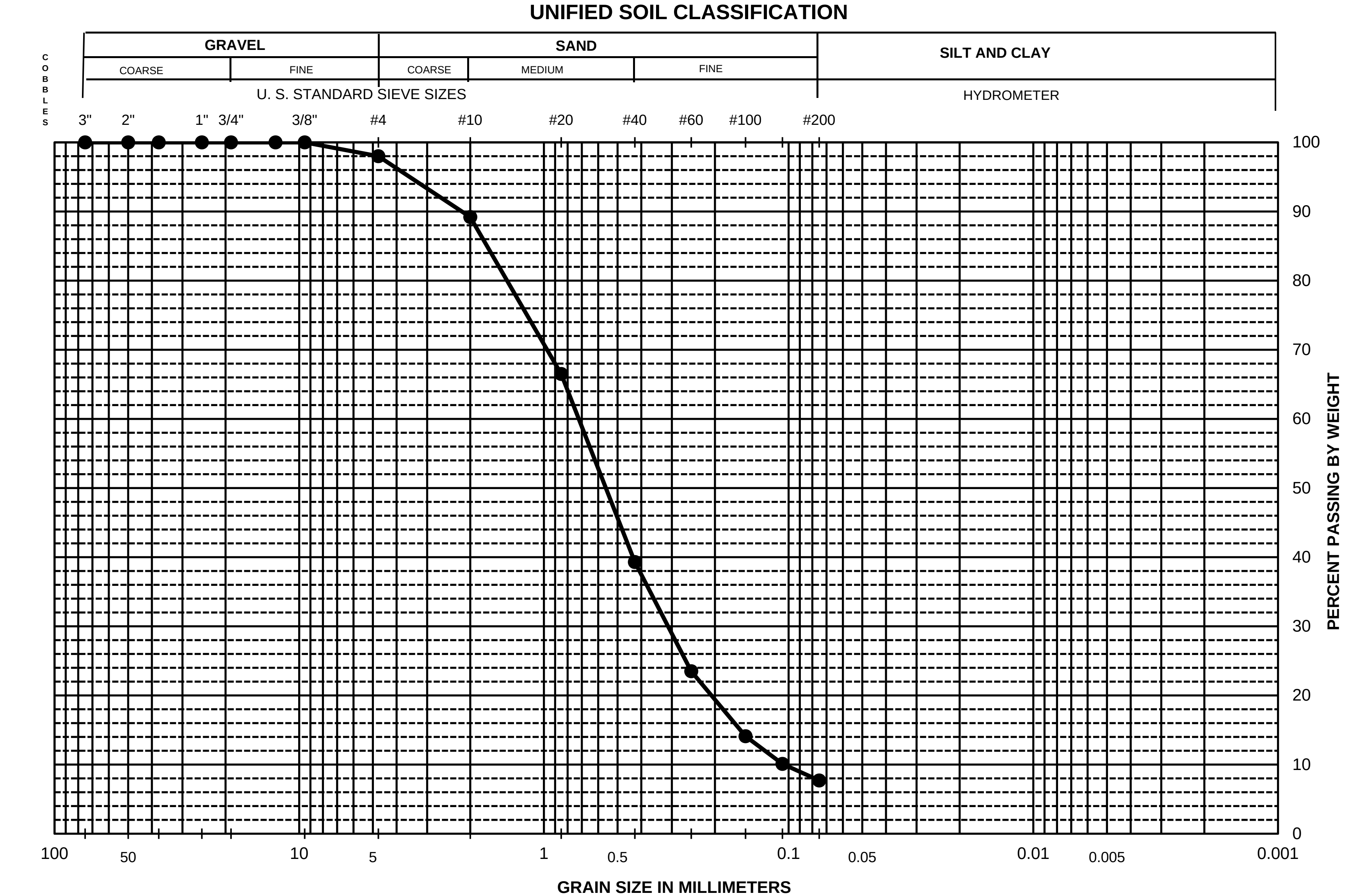
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-7

AECOM

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-3\Sieve MSWD Mission Springs B-3 20 ft.xlsx



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	100.0
#4	4.75	98.0
#10	2.00	89.2
#20	0.850	66.5
#40	0.425	39.3
#60	0.250	23.5
#100	0.150	14.1
#140	0.106	10.1
#200	0.075	7.7
Hydrometer Analysis		

% Cobbles	---
% Gravel	2.0
% Sand	90.3
% Fines	7.7
D_{85}	1.707
D_{60}	0.720
D_{50}	0.558
D_{30}	0.311
D_{15}	0.158
D_{10}	0.104
C_u	6.9
C_c	1.3

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-4	2	5.0	●	0.8	---	---	---	**Grayish brown Well Graded SAND with Silt (SW-SM)**

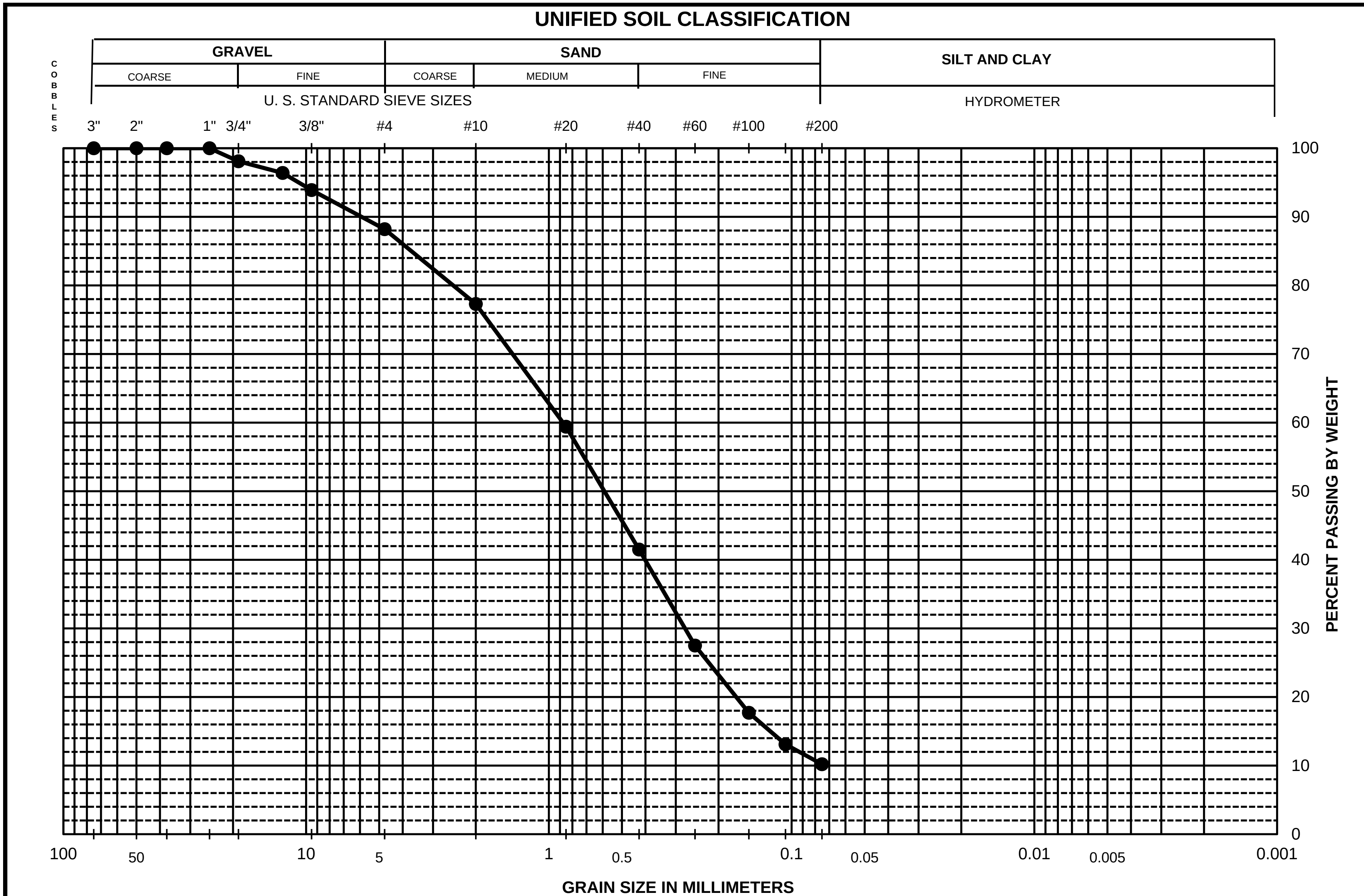
PROJECT NAME: **MSWD Mission Springs**

PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-8

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	98.1
1/2"	12.50	96.4
3/8"	9.50	93.9
#4	4.75	88.2
#10	2.00	77.3
#20	0.850	59.4
#40	0.425	41.5
#60	0.250	27.5
#100	0.150	17.7
#140	0.106	13.1
#200	0.075	10.2
Hydrometer Analysis		

% Cobbles	---
% Gravel	11.8
% Sand	78.0
% Fines	10.2
D_{85}	3.685
D_{60}	0.875
D_{50}	0.591
D_{30}	0.275
D_{15}	0.122
D_{10}	----
C_u	----
C_c	-----

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-4	5	25.0	●	0.7	---	---	---	**Light brownish gray Poorly Graded SAND with Silt (SP-SM)**

PROJECT NAME: **MSWD Mission Springs**

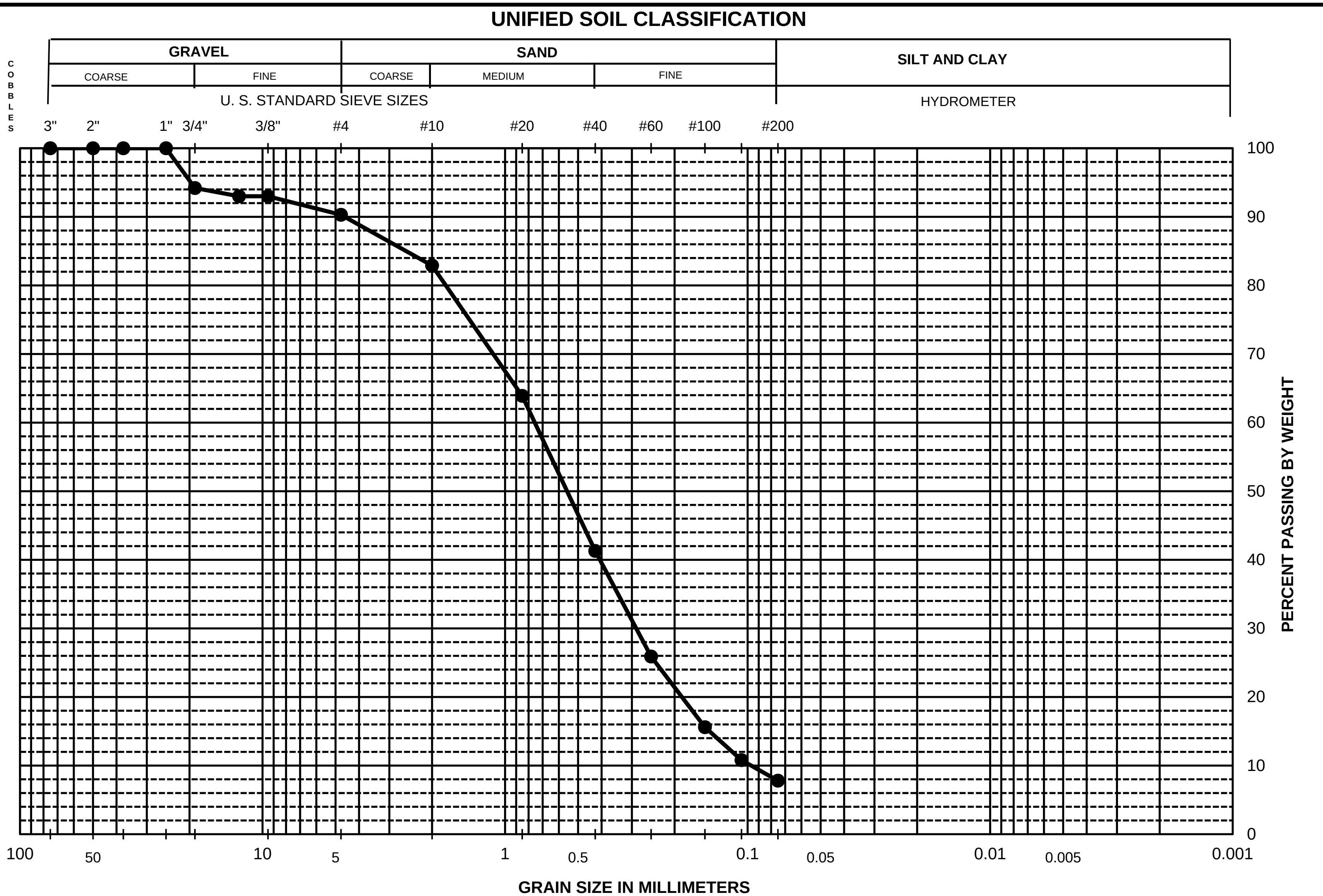
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-9

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-4\Sieve MSWD Mission Springs B-4 25 ft.xlsx

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	94.2
1/2"	12.50	93.0
3/8"	9.50	93.0
#4	4.75	90.3
#10	2.00	82.9
#20	0.850	63.9
#40	0.425	41.3
#60	0.250	25.9
#100	0.150	15.6
#140	0.106	10.8
#200	0.075	7.8
Hydrometer Analysis		

% Cobbles	---
% Gravel	9.7
% Sand	82.5
% Fines	7.8
D_{85}	2.556
D_{60}	0.754
D_{50}	0.555
D_{30}	0.288
D_{15}	0.144
D_{10}	0.097
C_u	7.8
C_c	1.1

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-5	2	5.0	•	0.5	---	---	---	**Light brownish gray Well Graded SAND with Silt (SW-SM)**

PROJECT NAME: **MSWD Mission Springs**

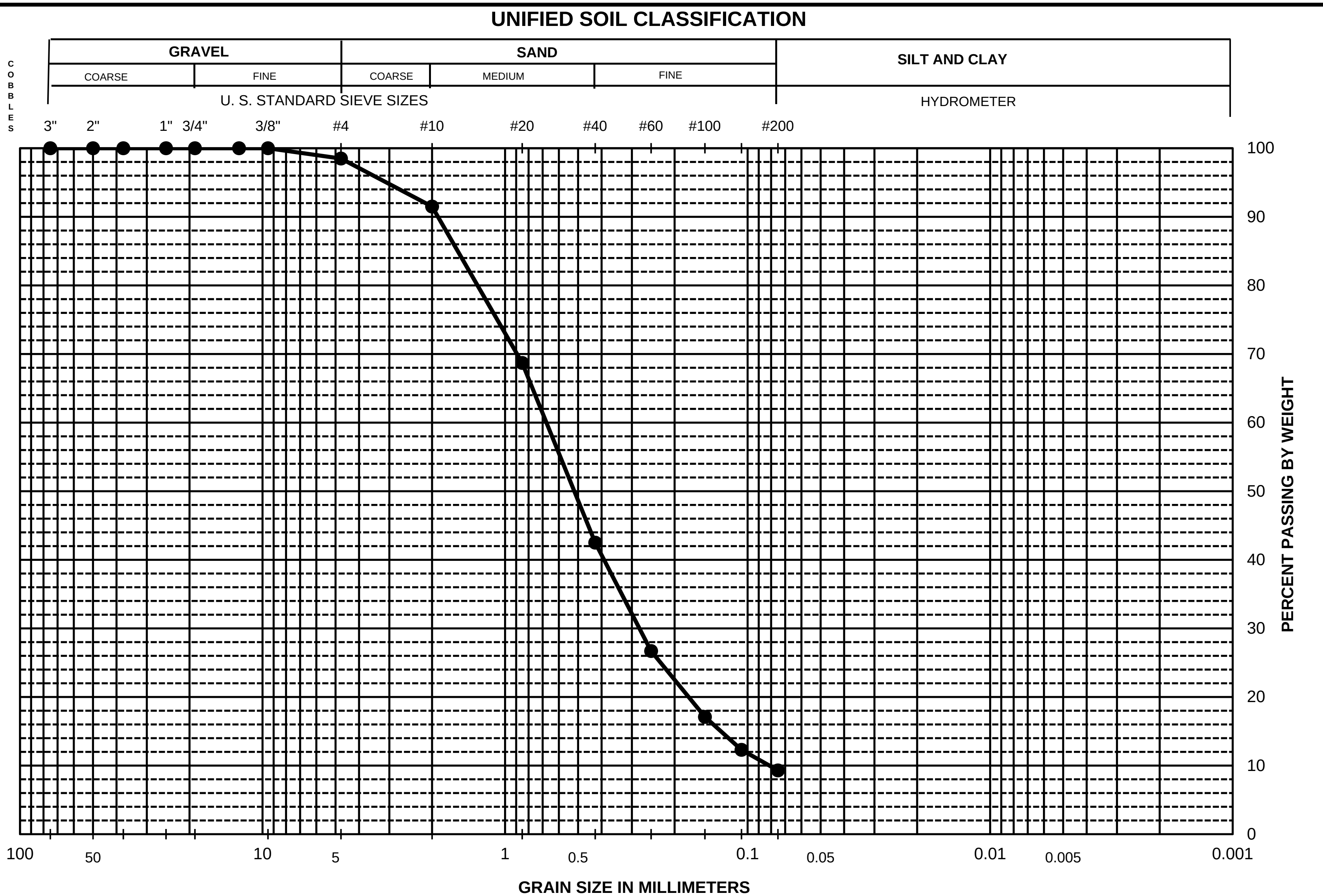
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-10

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-5\Sieve MSWD Mission Springs B-5 5 ft.xlsx

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	100.0
#4	4.75	98.5
#10	2.00	91.5
#20	0.850	68.7
#40	0.425	42.5
#60	0.250	26.7
#100	0.150	17.1
#140	0.106	12.3
#200	0.075	9.3
Hydrometer Analysis		

% Cobbles	---
% Gravel	1.5
% Sand	89.2
% Fines	9.3
D_{85}	1.567
D_{60}	0.675
D_{50}	0.518
D_{30}	0.279
D_{15}	0.129
D_{10}	0.081
C_u	8.3
C_c	1.4

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-6	1	0~5	●	0.8	---	---	---	**Light olive brown Well Graded SAND with Silt (SW-SM)**

PROJECT NAME: **MSWD Mission Springs**

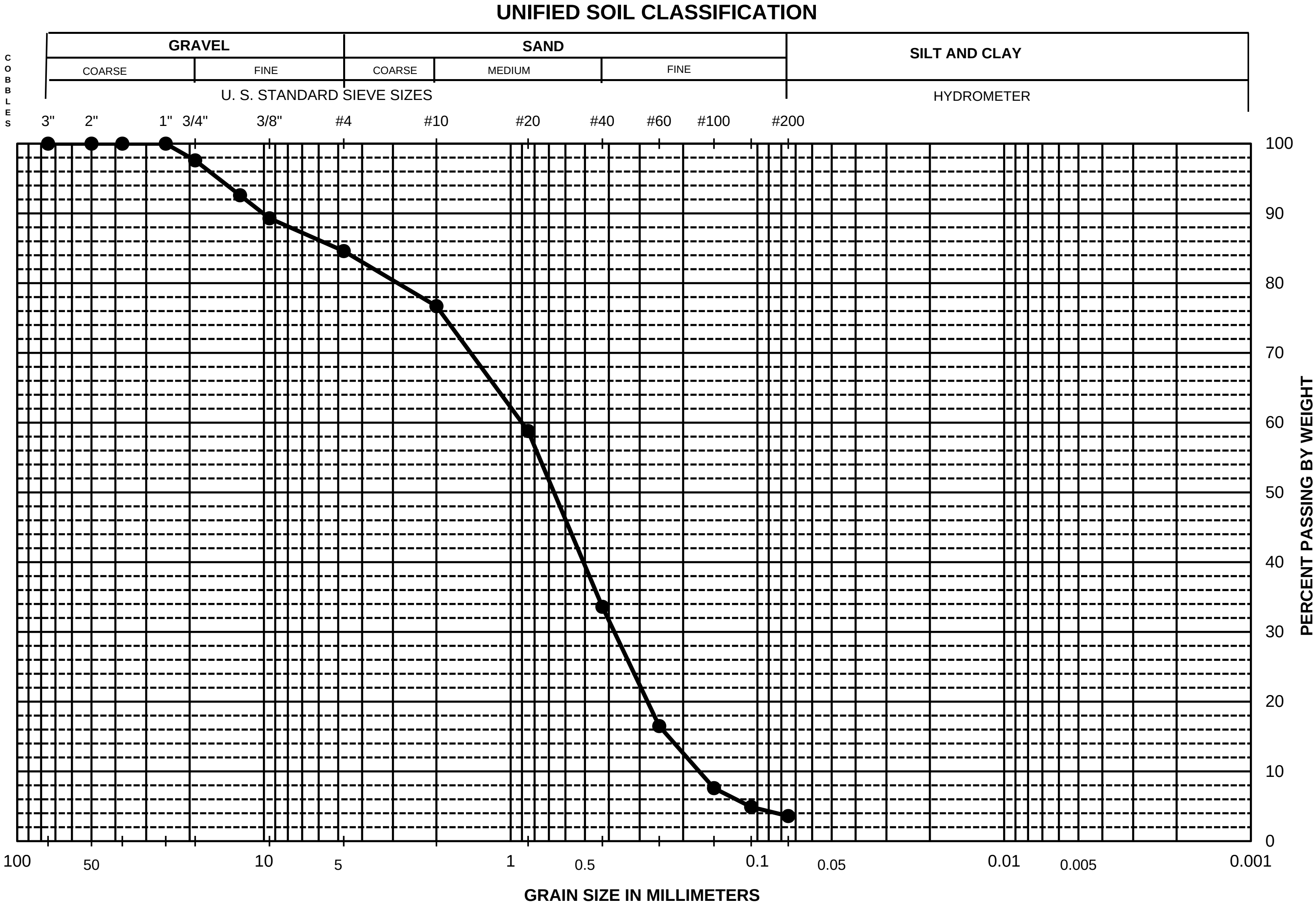
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-11

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-6\Sieve MSWD Mission Springs B-6 0-5 ft.xlsx

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	97.6
1/2"	12.50	92.6
3/8"	9.50	89.3
#4	4.75	84.6
#10	2.00	76.7
#20	0.850	58.8
#40	0.425	33.6
#60	0.250	16.5
#100	0.150	7.6
#140	0.106	4.9
#200	0.075	3.6
Hydrometer Analysis		

% Cobbles	---
% Gravel	15.4
% Sand	81.0
% Fines	3.6
D_{85}	5.039
D_{60}	0.900
D_{50}	0.667
D_{30}	0.380
D_{15}	0.229
D_{10}	0.172
C_u	5.2
C_c	0.9

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-6	3	15.0	●	0.7	---	---	---	**Light brownish gray Poorly Graded SAND with Gravel (SP)**

PROJECT NAME: MSWD Mission Springs

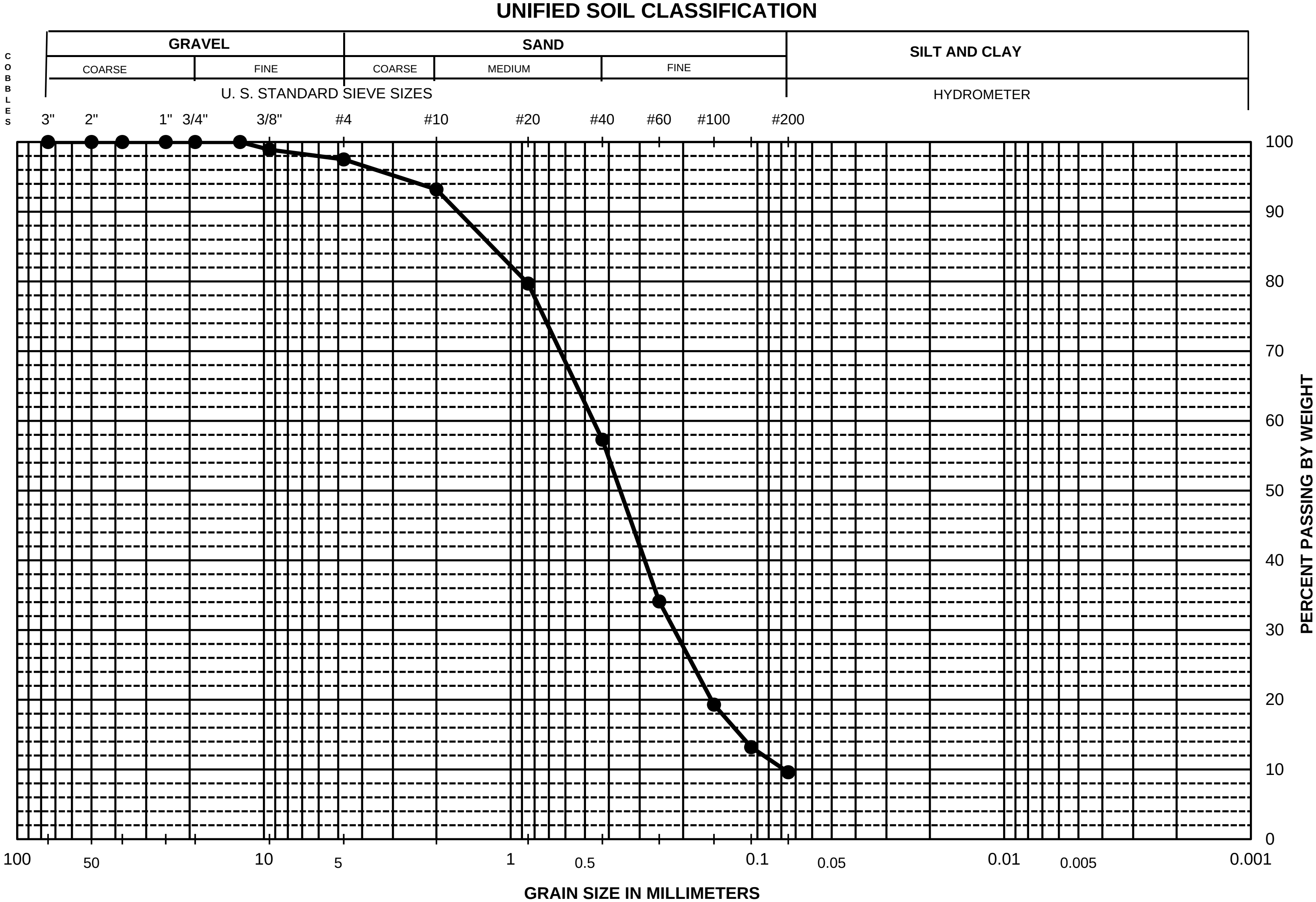
PROJECT NUMBER: 60551186

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-12

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-6\Sieve MSWD Mission Springs B-6 15 ft.xlsx

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	98.9
#4	4.75	97.5
#10	2.00	93.2
#20	0.850	79.7
#40	0.425	57.3
#60	0.250	34.1
#100	0.150	19.3
#140	0.106	13.2
#200	0.075	9.6
Hydrometer Analysis		

% Cobbles	---
% Gravel	2.5
% Sand	87.9
% Fines	9.6
D_{85}	1.189
D_{60}	0.462
D_{50}	0.360
D_{30}	0.217
D_{15}	0.117
D_{10}	0.078
C_u	5.9
C_c	1.3

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-6	5	25.0	●	0.7	---	---	---	**Grayish brown Poorly Graded SAND with Silt (SP-SM)**

PROJECT NAME: **MSWD Mission Springs**

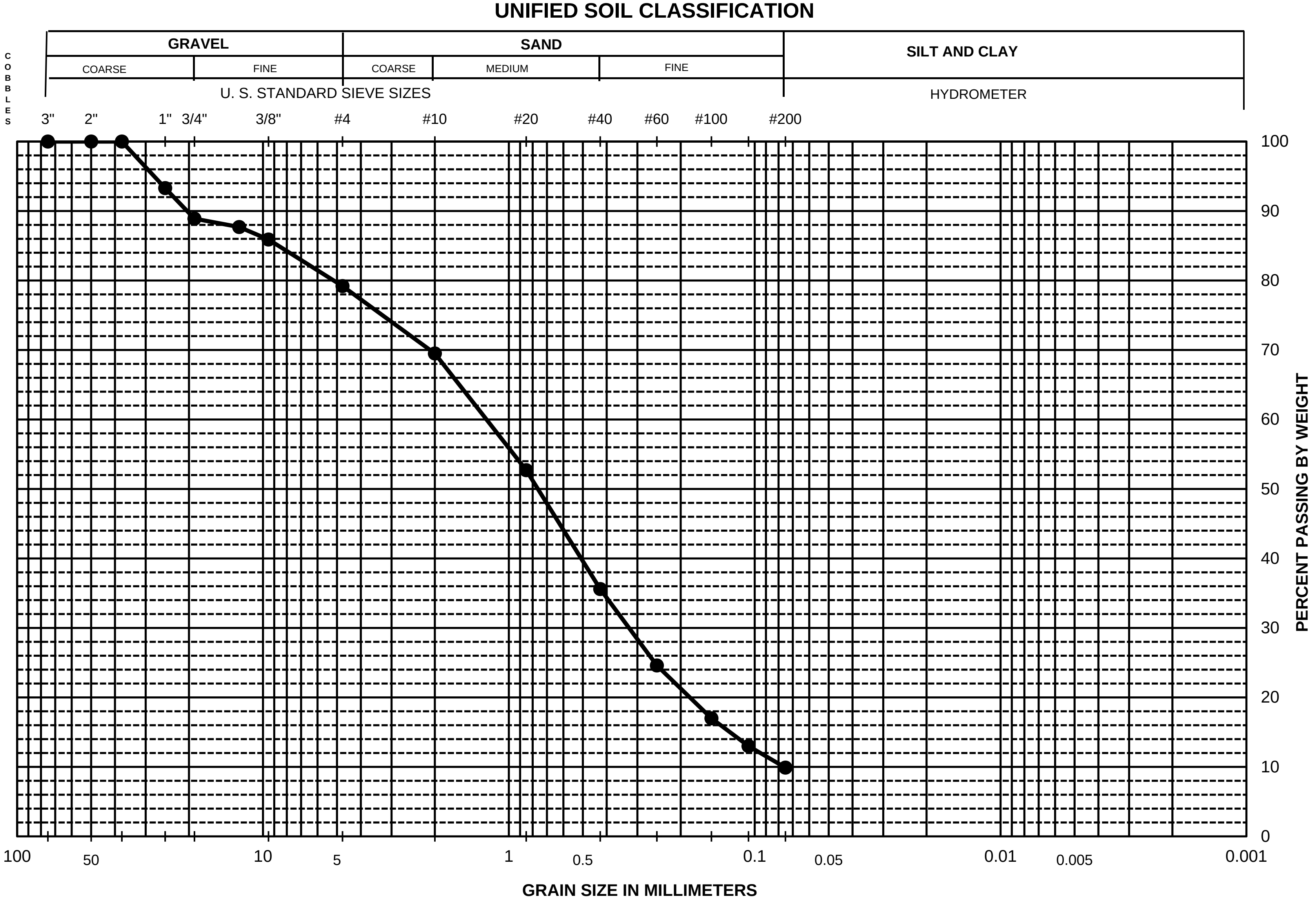
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-13

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-6\Sieve MSWD Mission Springs B-6 25 ft.xlsx

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	93.3
3/4"	19.00	88.9
1/2"	12.50	87.7
3/8"	9.50	85.9
#4	4.75	79.2
#10	2.00	69.5
#20	0.850	52.7
#40	0.425	35.6
#60	0.250	24.6
#100	0.150	17.0
#140	0.106	13.0
#200	0.075	9.9

Hydrometer Analysis

% Cobbles	---
% Gravel	20.8
% Sand	69.3
% Fines	9.9
D_{85}	8.655
D_{60}	1.233
D_{50}	0.762
D_{30}	0.324
D_{15}	0.126
D_{10}	0.076
C_u	16.3
C_c	1.1

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-6	8B	41.0	●	0.8	---	---	---	**Light olive brown Well Graded SAND with Silt and Gravel (SW-SM)**

PROJECT NAME: **MSWD Mission Springs**

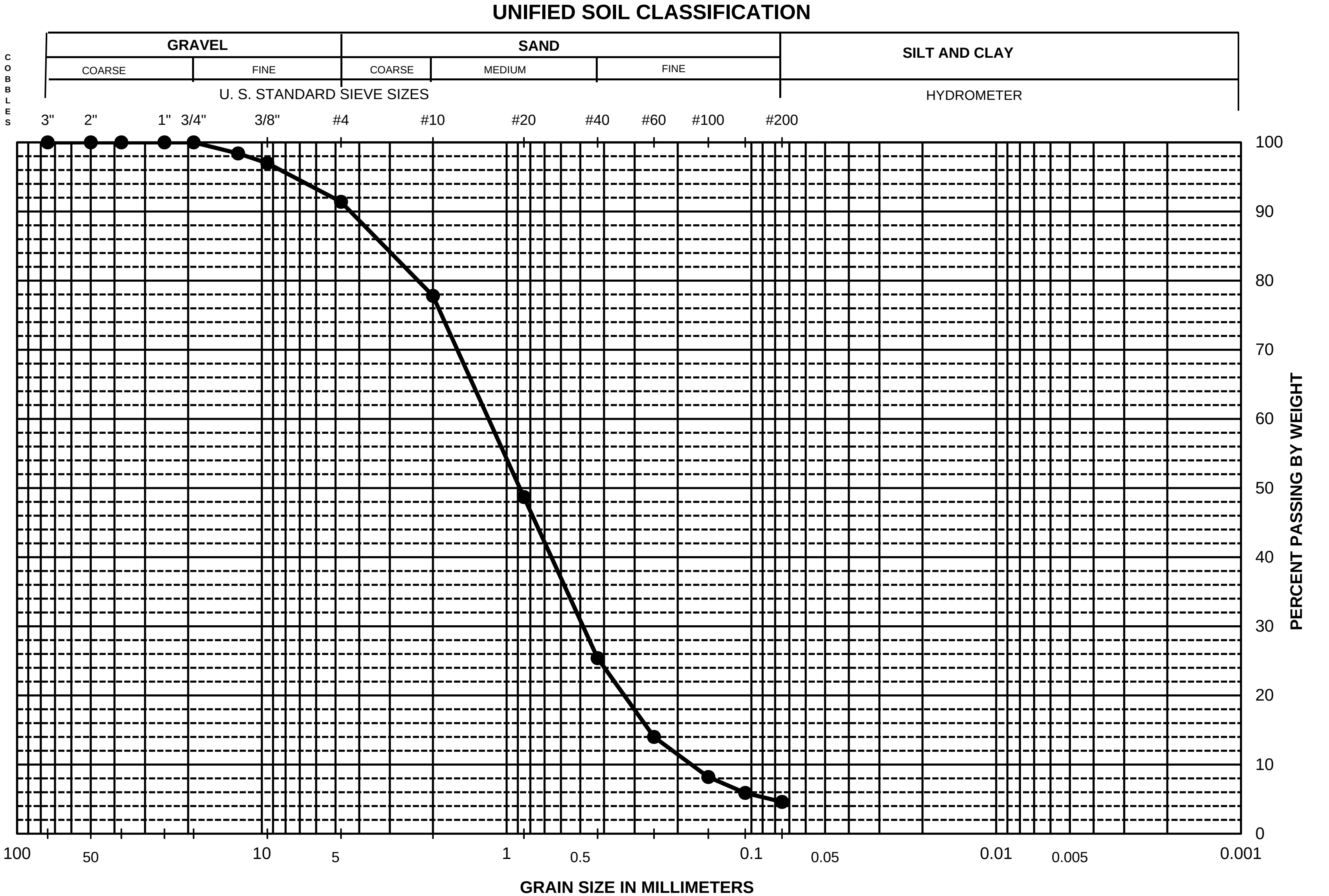
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-14

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-6\Sieve MSWD Mission Springs B-6 41 ft.xlsx

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	98.4
3/8"	9.50	97.0
#4	4.75	91.4
#10	2.00	77.8
#20	0.850	48.7
#40	0.425	25.4
#60	0.250	14.0
#100	0.150	8.2
#140	0.106	5.9
#200	0.075	4.6
Hydrometer Analysis		

% Cobbles	---
% Gravel	8.6
% Sand	86.8
% Fines	4.6
D_{85}	3.162
D_{60}	1.185
D_{50}	0.883
D_{30}	0.487
D_{15}	0.262
D_{10}	0.176
C_u	6.7
C_c	1.1

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-7	3	10.0	●	0.6	---	---	---	**Light brownish gray Well Graded SAND (SW)**

PROJECT NAME: **MSWD Mission Springs**

PROJECT NUMBER: **60551186**

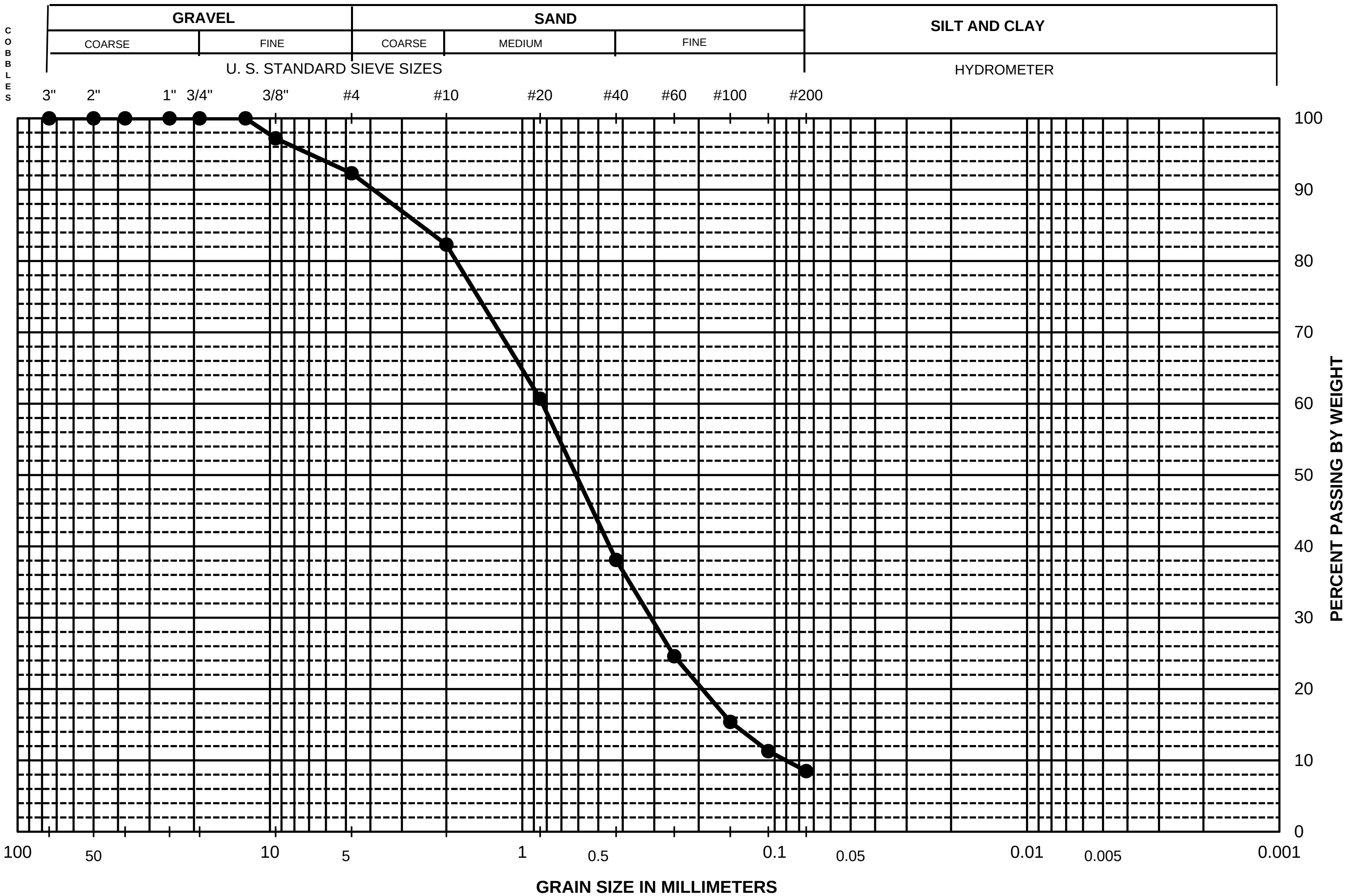
PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-15

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-7\Sieve MSWD Mission Springs B-7 10 ft.xlsx

AECOM

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	97.2
#4	4.75	92.3
#10	2.00	82.3
#20	0.850	60.7
#40	0.425	38.1
#60	0.250	24.6
#100	0.150	15.4
#140	0.106	11.3
#200	0.075	8.5

Hydrometer Analysis

% Cobbles	---
% Gravel	7.7
% Sand	83.8
% Fines	8.5
D_{85}	2.526
D_{60}	0.832
D_{50}	0.612
D_{30}	0.309
D_{15}	0.145
D_{10}	0.090
C_u	9.2
C_c	1.3

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-7	5	20.0	●	0.8	---	---	---	**Light brownish gray Well Graded SAND with Silt (SW-SM)**

PROJECT NAME: **MSWD Mission Springs**

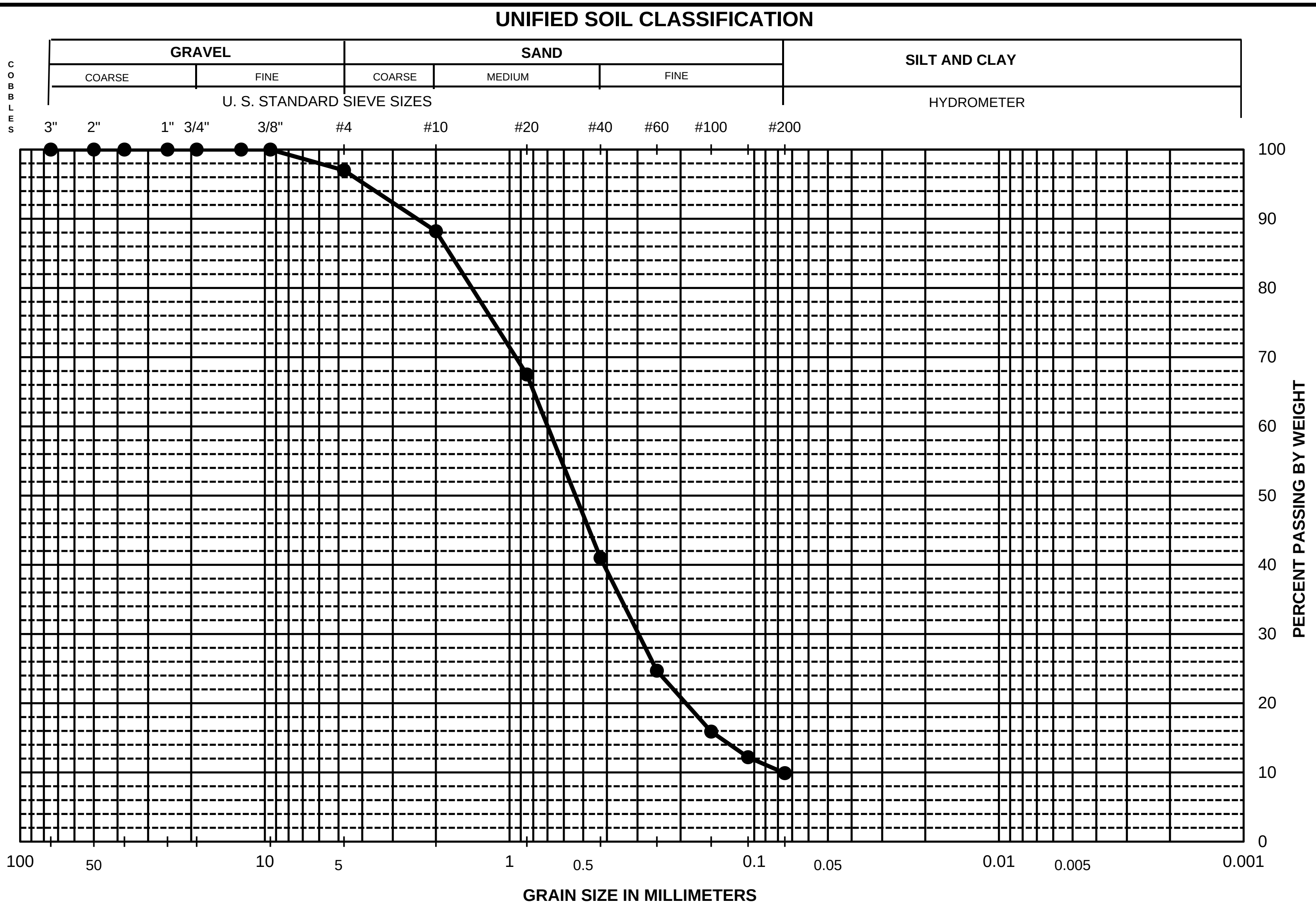
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-16

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-7\Sieve MSWD Mission Springs B-7 20 ft.xlsx

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	100.0
#4	4.75	97.0
#10	2.00	88.2
#20	0.850	67.5
#40	0.425	41.0
#60	0.250	24.7
#100	0.150	15.9
#140	0.106	12.2
#200	0.075	9.9
Hydrometer Analysis		

% Cobbles	---
% Gravel	3.0
% Sand	87.1
% Fines	9.9
D_{85}	1.752
D_{60}	0.699
D_{50}	0.538
D_{30}	0.297
D_{15}	0.138
D_{10}	0.076
C_u	9.2
C_c	1.7

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-8	1	0~5	●	0.6	---	---	---	**Grayish brown Well Graded SAND with Silt (SW-SM)**

PROJECT NAME: **MSWD Mission Springs**

PROJECT NUMBER: **60551186**

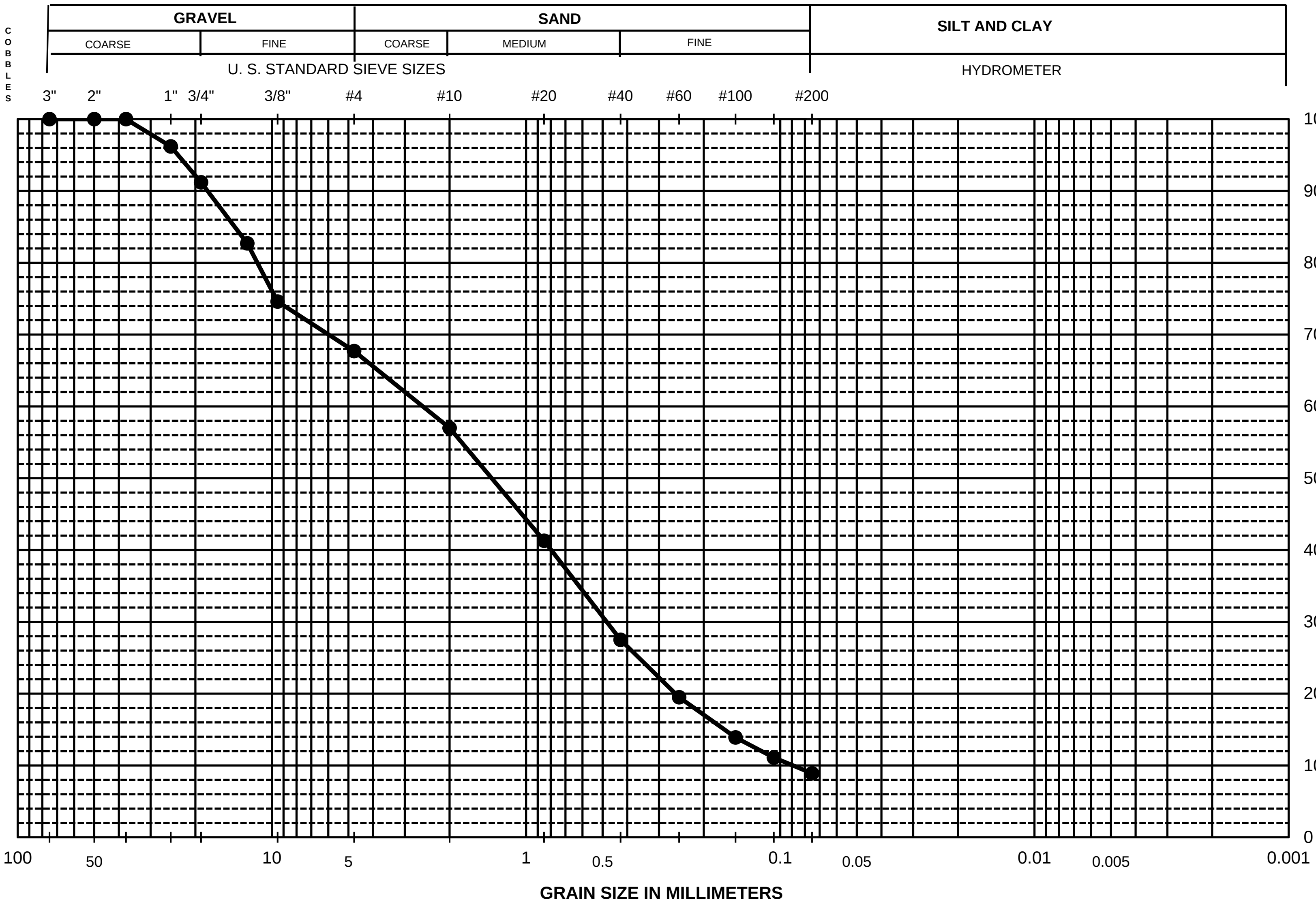
PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-17

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-8\Sieve MSWD Mission Springs B-8 0-5 ft.xlsx

AECOM

UNIFIED SOIL CLASSIFICATION



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	96.2
3/4"	19.00	91.2
1/2"	12.50	82.7
3/8"	9.50	74.6
#4	4.75	67.7
#10	2.00	57.0
#20	0.850	41.3
#40	0.425	27.5
#60	0.250	19.5
#100	0.150	13.9
#140	0.106	11.1
#200	0.075	8.9
Hydrometer Analysis		

% Cobbles	---
% Gravel	32.3
% Sand	58.8
% Fines	8.9
D_{85}	14.000
D_{60}	2.549
D_{50}	1.366
D_{30}	0.482
D_{15}	0.166
D_{10}	0.089
C_u	28.6
C_c	1.0

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-8	3	10.0	●	0.3	---	---	---	**Grayish brown Well Graded SAND with Silt and Gravel (SW-SM)**

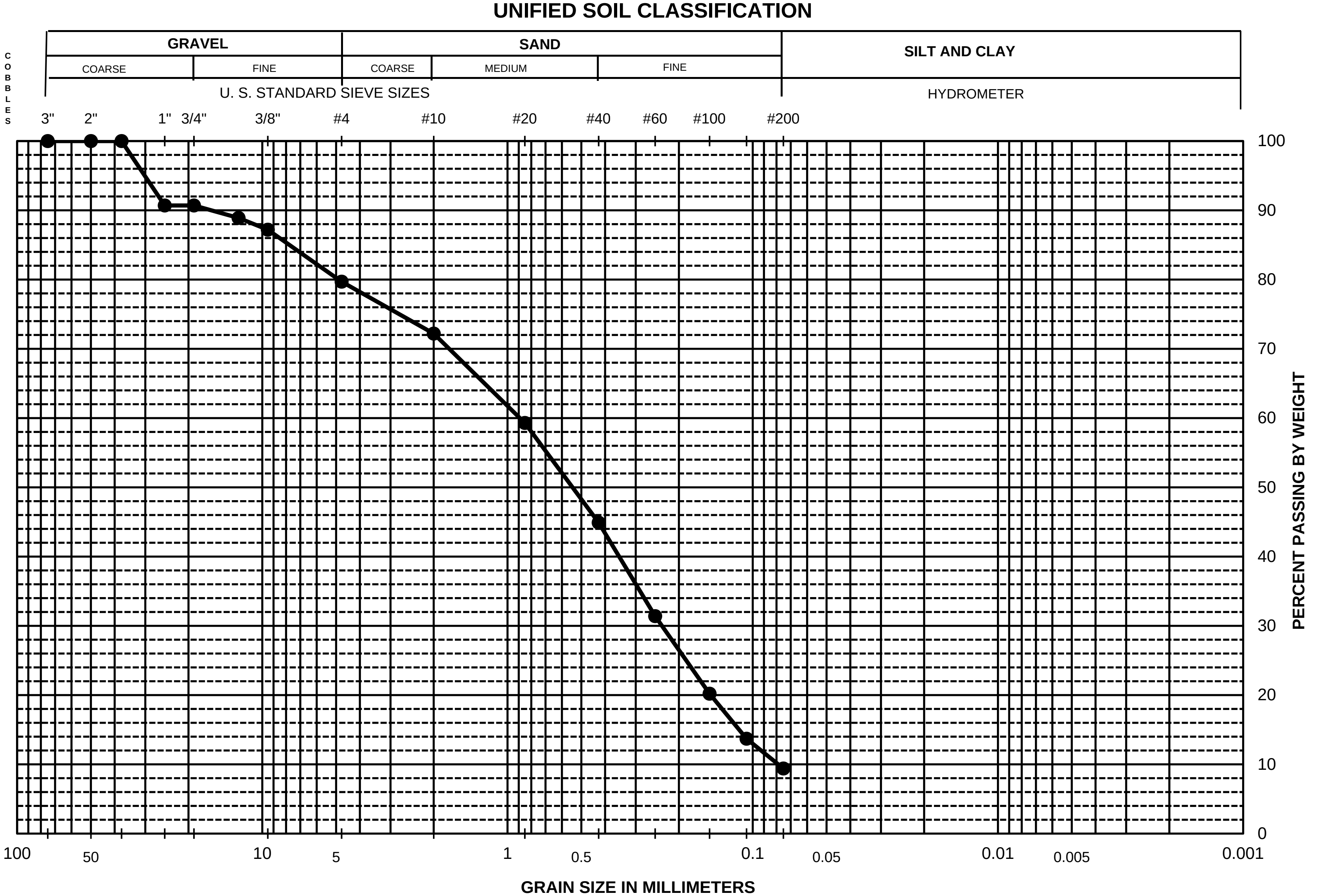
PROJECT NAME: **MSWD Mission Springs**

PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-18

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	90.7
3/4"	19.00	90.7
1/2"	12.50	88.9
3/8"	9.50	87.2
#4	4.75	79.7
#10	2.00	72.2
#20	0.850	59.3
#40	0.425	44.9
#60	0.250	31.4
#100	0.150	20.2
#140	0.106	13.7
#200	0.075	9.4

Hydrometer Analysis

% Cobbles	---
% Gravel	20.3
% Sand	70.3
% Fines	9.4
D_{85}	7.752
D_{60}	0.890
D_{50}	0.543
D_{30}	0.235
D_{15}	0.114
D_{10}	0.079
C_u	11.3
C_c	0.8

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-9	2	5.0	●	0.2	---	---	---	**Brownish gray Poorly Graded SAND with Silt and Gravel (SP-SM)**

PROJECT NAME: **MSWD Mission Springs**

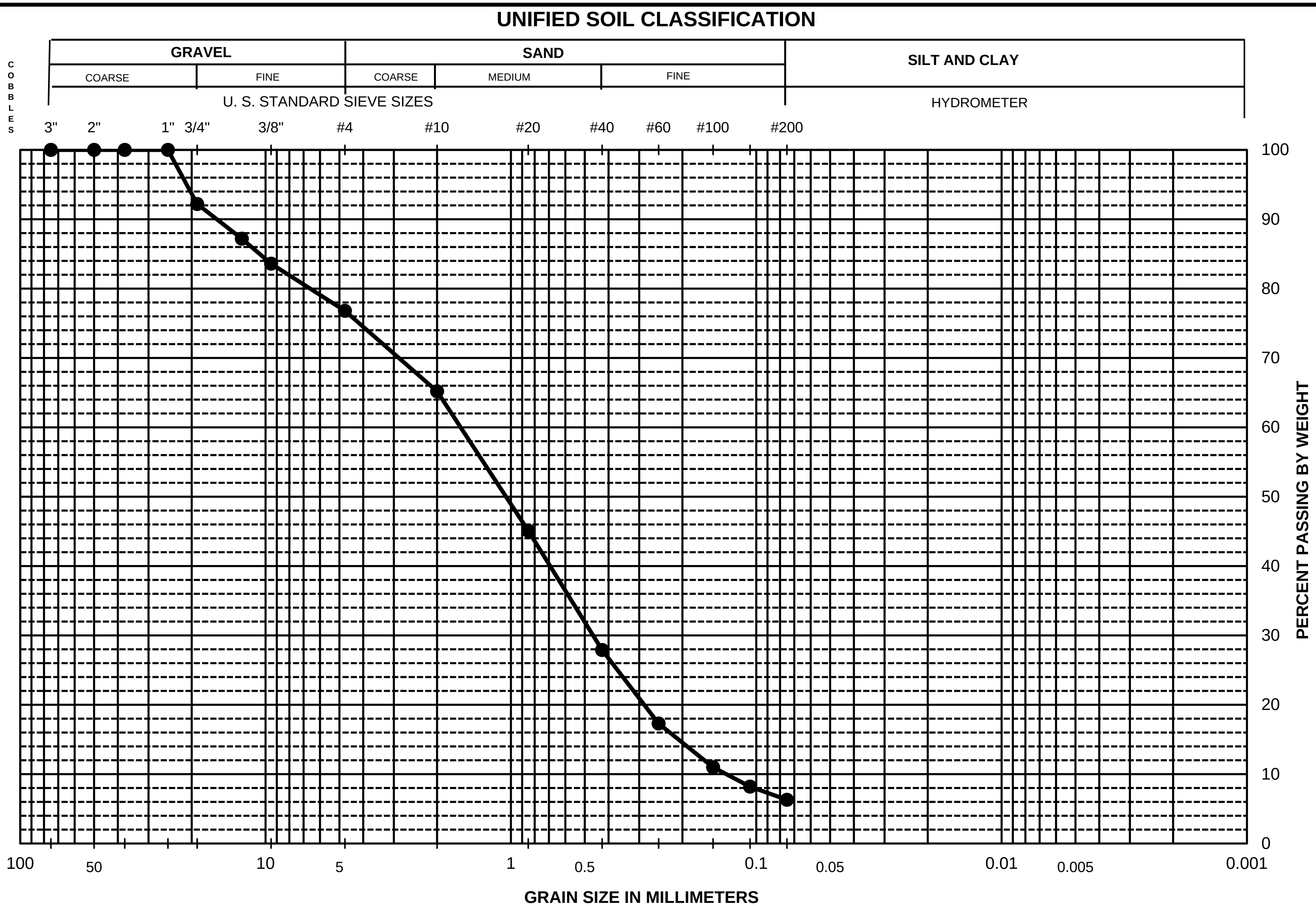
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-19

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-9\Sieve MSWD Mission Springs B-9 5 ft.xlsx

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	92.2
1/2"	12.50	87.2
3/8"	9.50	83.6
#4	4.75	76.8
#10	2.00	65.2
#20	0.850	45.1
#40	0.425	27.9
#60	0.250	17.3
#100	0.150	11.0
#140	0.106	8.2
#200	0.075	6.3
Hydrometer Analysis		

% Cobbles	---
% Gravel	23.2
% Sand	70.5
% Fines	6.3
D_{85}	10.570
D_{60}	1.603
D_{50}	1.047
D_{30}	0.463
D_{15}	0.207
D_{10}	0.133
C_u	12.1
C_c	1.0

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-9	7	30.0	●	0.5	---	---	---	**Brownish gray Well Graded SAND with Silt and Gravel (SW-SM)**

PROJECT NAME: **MSWD Mission Springs**

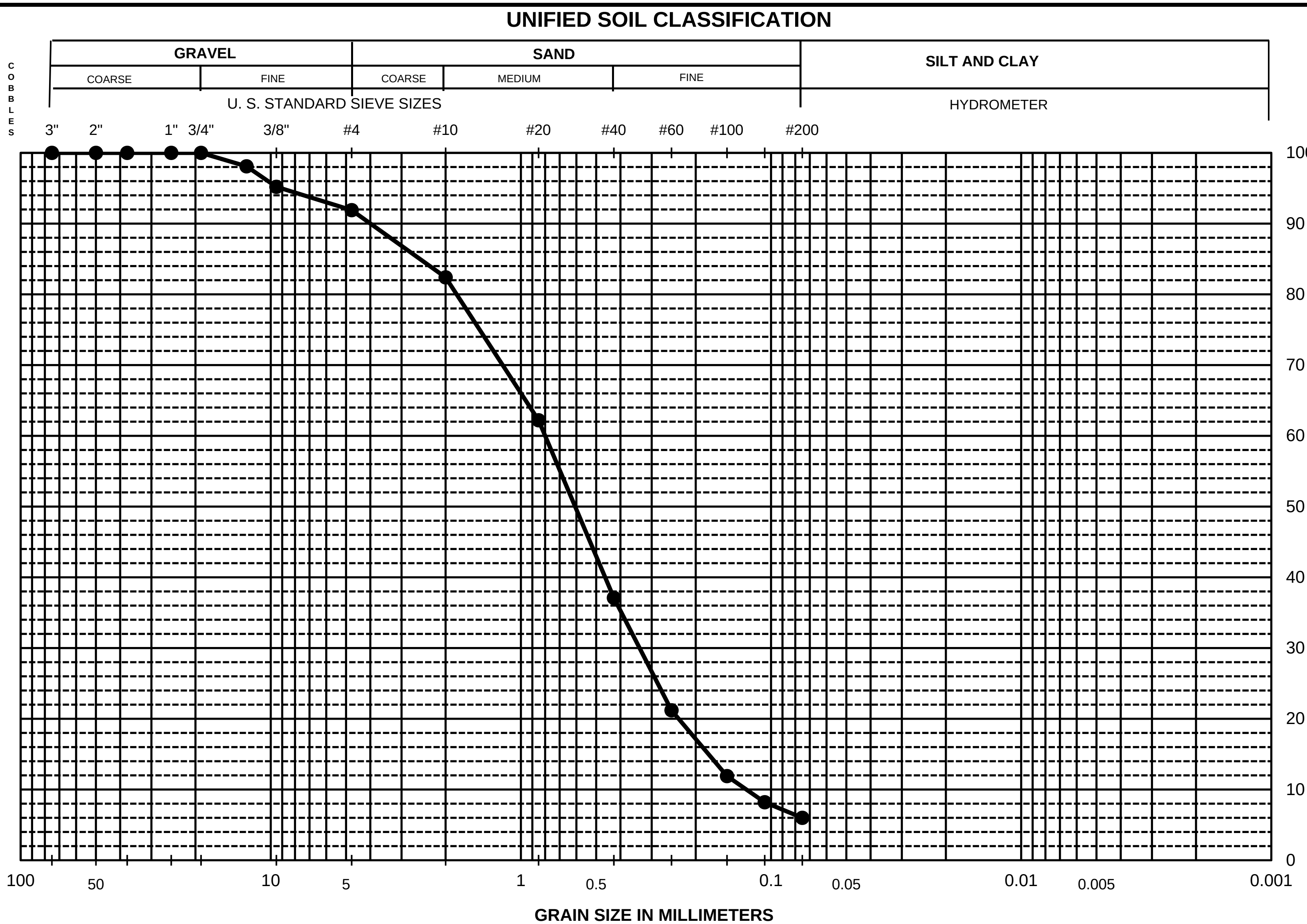
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-20

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-9\Sieve MSWD Mission Springs B-9 30 ft.xlsx

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	98.1
3/8"	9.50	95.2
#4	4.75	91.9
#10	2.00	82.4
#20	0.850	62.2
#40	0.425	37.1
#60	0.250	21.2
#100	0.150	11.9
#140	0.106	8.2
#200	0.075	6.0
Hydrometer Analysis		

% Cobbles	---
% Gravel	8.1
% Sand	85.9
% Fines	6.0
D_{85}	2.534
D_{60}	0.800
D_{50}	0.607
D_{30}	0.335
D_{15}	0.178
D_{10}	0.126
C_u	6.4
C_c	1.1

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-9	9	45.0	●	0.6	---	---	---	**Brownish gray Well Graded SAND with Silt (SW-SM)**

PROJECT NAME: **MSWD Mission Springs**

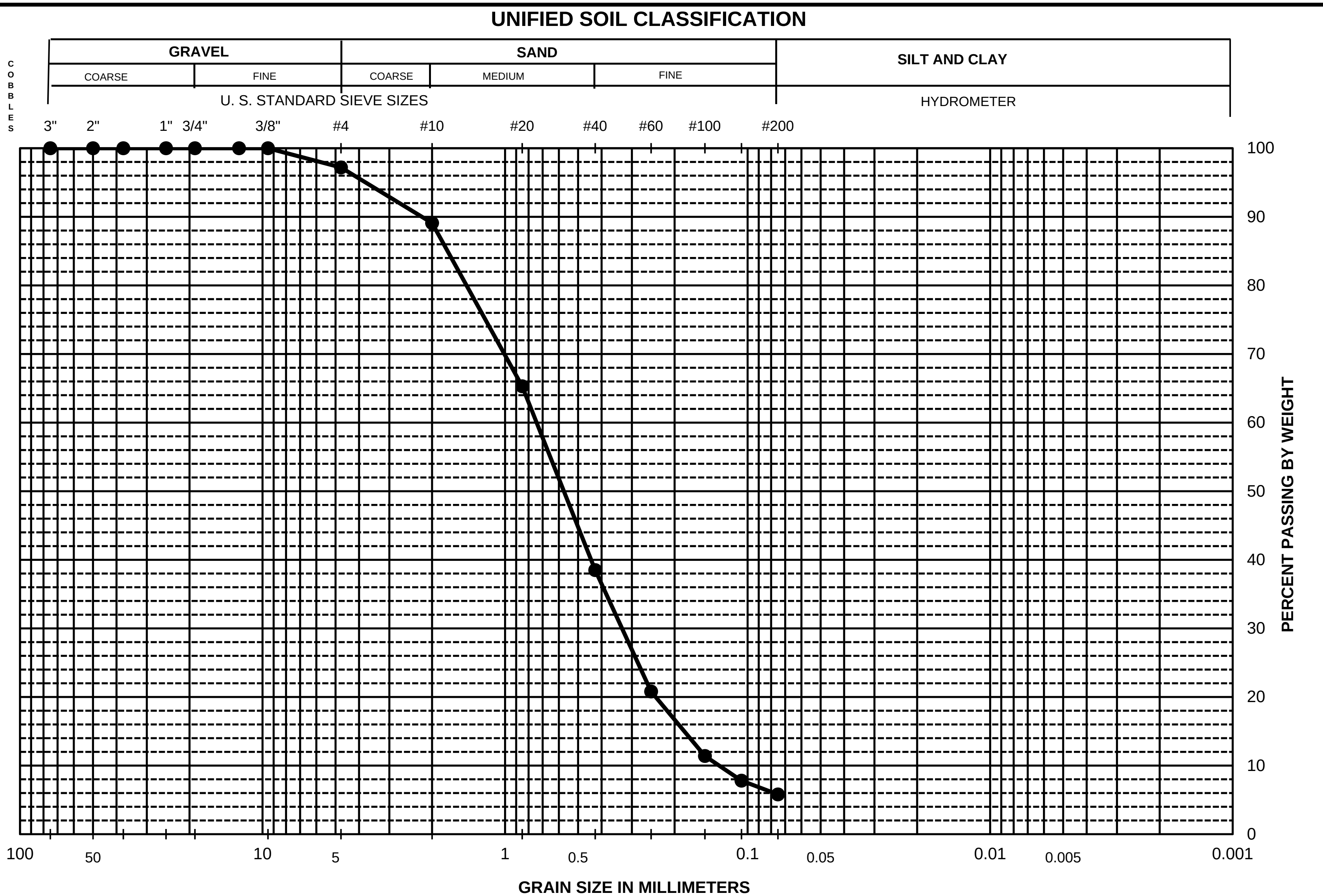
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-21

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-9\Sieve MSWD Mission Springs B-9 45 ft.xlsx

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	100.0
3/8"	9.50	100.0
#4	4.75	97.2
#10	2.00	89.1
#20	0.850	65.3
#40	0.425	38.5
#60	0.250	20.8
#100	0.150	11.4
#140	0.106	7.8
#200	0.075	5.8
Hydrometer Analysis		

% Cobbles	---
% Gravel	2.8
% Sand	91.4
% Fines	5.8
D_{85}	1.726
D_{60}	0.741
D_{50}	0.572
D_{30}	0.329
D_{15}	0.182
D_{10}	0.131
C_u	5.7
C_c	1.1

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 µm	Description and Classification
B-10	1	0~5	●	0.4	---	---	---	**Brownish gray Poorly Graded SAND with Silt (SP-SM)**

PROJECT NAME: **MSWD Mission Springs**

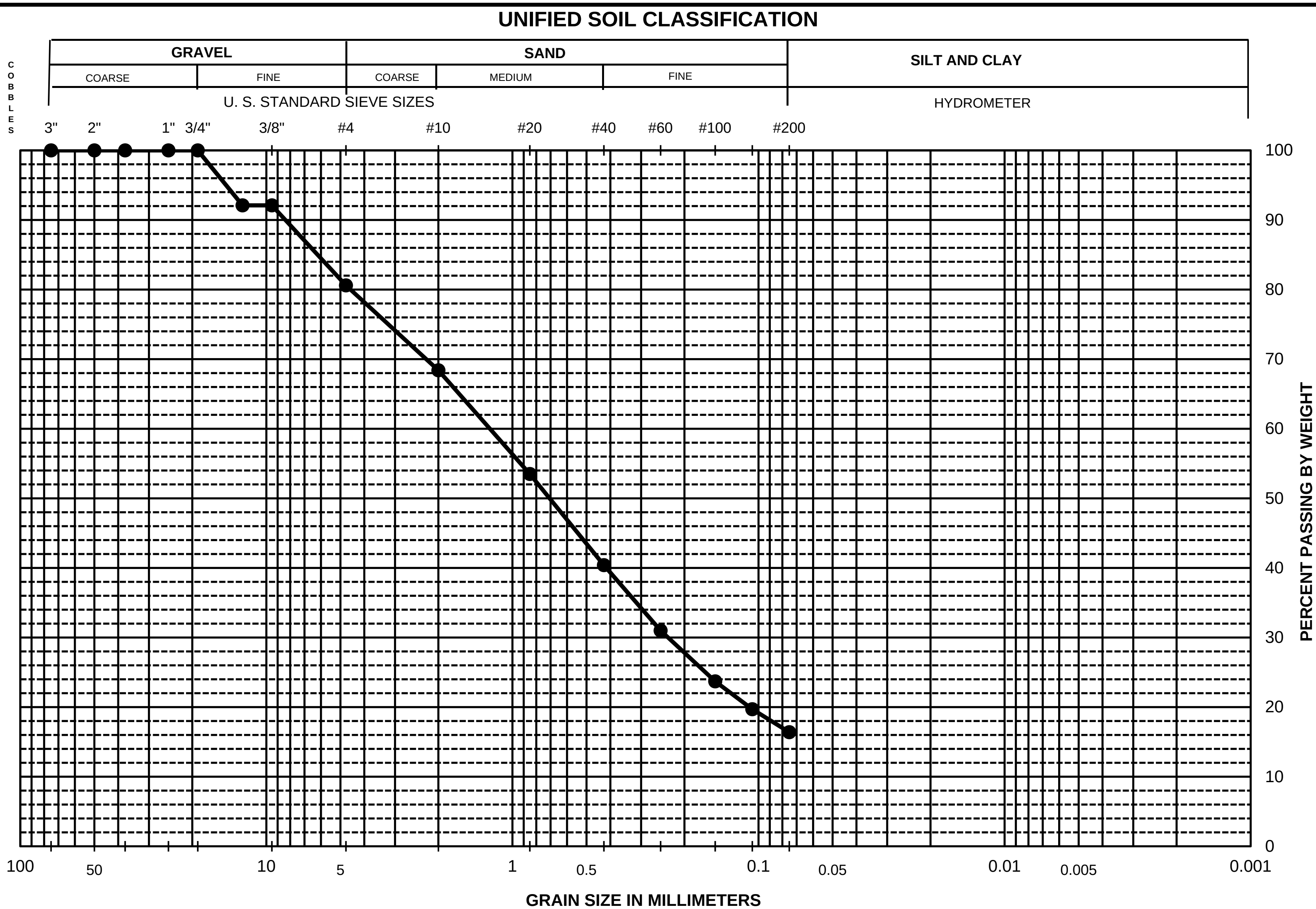
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-22

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-10\Sieve MSWD Mission Springs B-10 0-5 ft.xlsx

AECOM



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	92.1
3/8"	9.50	92.1
#4	4.75	80.6
#10	2.00	68.4
#20	0.850	53.5
#40	0.425	40.4
#60	0.250	31.0
#100	0.150	23.7
#140	0.106	19.7
#200	0.075	16.4
Hydrometer Analysis		

% Cobbles	---
% Gravel	19.4
% Sand	64.2
% Fines	16.4
D_{85}	6.878
D_{60}	1.235
D_{50}	0.706
D_{30}	0.233
D_{15}	----
D_{10}	-----
C_u	----
C_c	-----

Boring No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 2 μm	Description and Classification
B-10	3	10.0	●	0.3	---	---	---	**Brownish gray Silty SAND with Gravel (SM)**

PROJECT NAME: **MSWD Mission Springs**

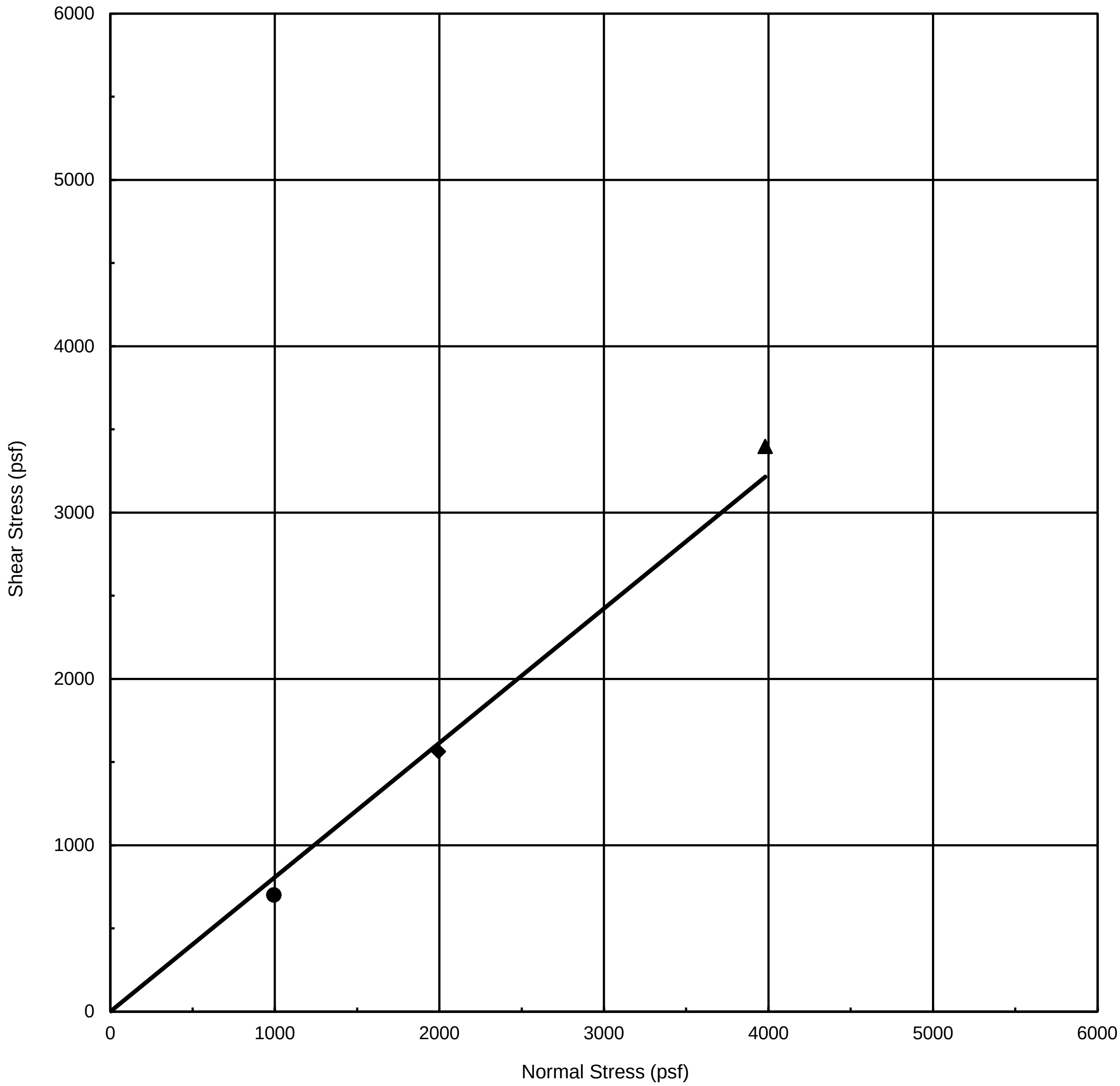
PROJECT NUMBER: **60551186**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure C-23

Q:\DCS\Projects\WTR\60551186_MSWD_RWWTP\400-Technical\432_Geotech\Lab Results\B-10\Sieve MSWD Mission Springs B-10 10 ft.xlsx

AECOM



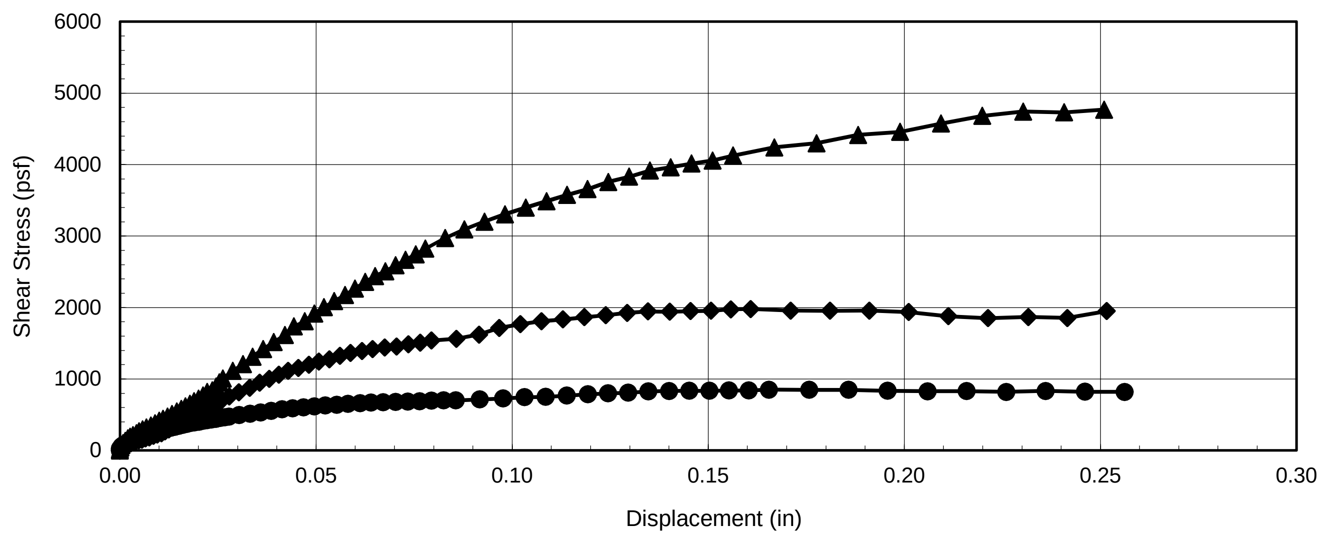
Peak Values are : ● ,solid trend line							Ultimate Values are: ❍ ,dashed trend line				
Exploration No.:	B-3			Strength Intercept (C) :	0.0	psf	Peak	XXXXX	psf	Ultimate	
Sample No.:	2				0.0	kPa		XXXXX	kPa		
Depth (ft | m)	5.0	1.5		Friction Angle (ϕ) :	41	degree		XXXXX	degree		
Description:	**Light brownish gray Well Graded SAND with Gravel (SW)**				Shear rate :		0.0050	(in/min) ,	0.0127	(cm/min)	
SYMBOL	% Water Content	Total Unit Weight (pcf)	Total Unit Weight (kN/m^3)	Dry Unit Weight (pcf)	Dry Unit Weight (kN/m^3)	Normal Stress (psf)	Normal Stress (kPa)	Peak Stress (psf)	Peak Stress (kPa)	Ultimate Stress (psf)	Ultimate Stress (kPa)
Initial / Set up	0.6	117.1	18.4	116.4	18.3	XXXXX	XXXXX	XXXXX	XXXXX	XXXXX	XXXXX
pre-shear ● spec. 1	25.0	132.5	20.8	106.0	16.7	994	48	702	34	XXXXX	XXXXX
pre-shear ◆ spec. 2	24.4	133.2	20.9	107.1	16.8	1995	96	1563	75	XXXXX	XXXXX
pre-shear ▲ spec. 3	24.1	133.6	21.0	107.7	16.9	3980	191	3398	163	XXXXX	XXXXX

AECOM	**MSWD Mission Springs** Project Number: 60551186 Test Date: 10/13/2017	**DIRECT SHEAR TEST** **ASTM D 3080**

Figure C-24

DIRECT SHEAR TEST
ASTM D 3080

Project Name :	MSWD Mission Springs	Boring No.:	B-3
Project Number :	60551186	Sample No.:	2
		Sample Depth (ft.):	5
Specimen Description :	Light brownish gray Well Graded SAND with Gravel (SW)		
		Normal Stress (psf):	994 ●
Apparatus No.:	DS3	Normal Stress (psf):	1995 ◆
Shear rate (in/min):	0.005	Normal Stress (psf):	3980 ▲



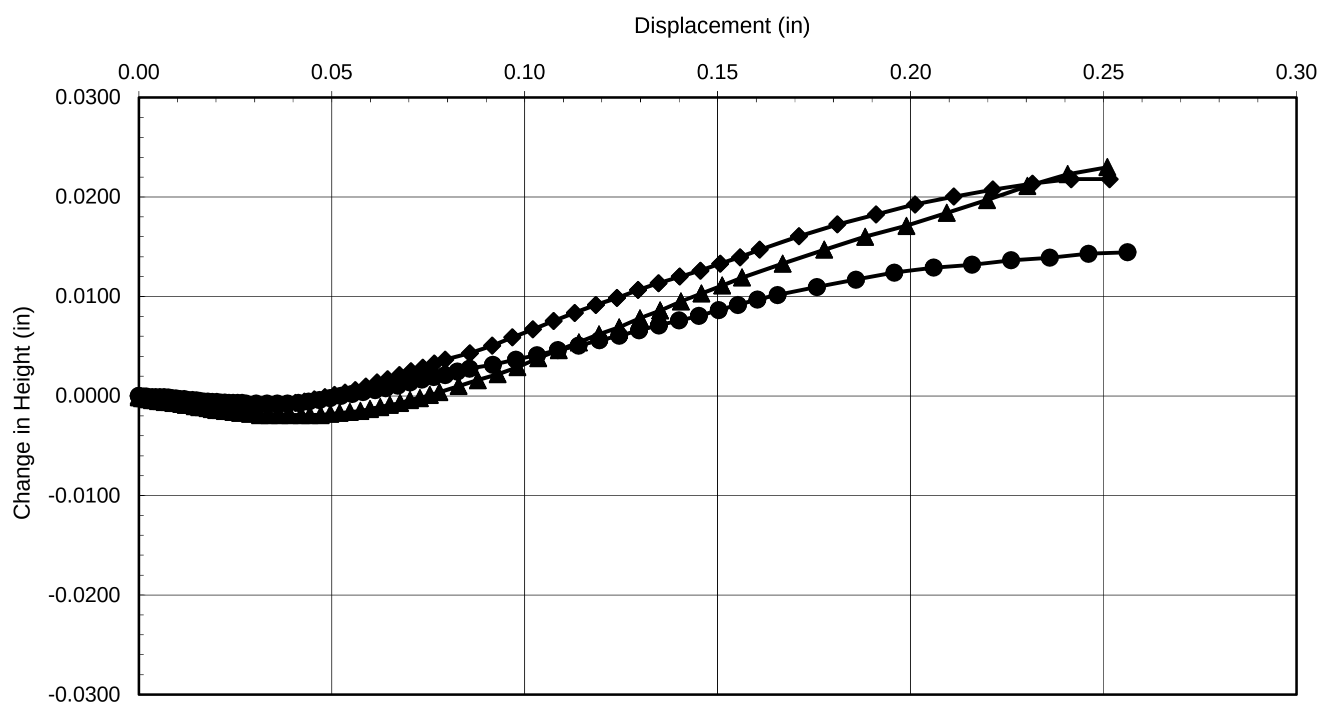


Table 1 - Laboratory Tests on Soil Samples

AECOM
MSWD Mission Springs
HDR Lab #17-0738LAB
2-Nov-17

Sample ID			B-3 Sample 3 @ 10' SP-SM	B-6 Sample 2 @ 10' GP-GM
Resistivity		**Units**		
as-received		ohm-cm	>4,400,000	>4,400,000
minimum		ohm-cm	9,600	2,400
pH			9.8	11.4
Electrical Conductivity		mS/cm	0.10	0.82
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	50	438
magnesium	Mg^{2+}	mg/kg	1.3	ND
sodium	Na^{1+}	mg/kg	29	31
potassium	K^{1+}	mg/kg	34	68
Anions				
hydroxide	OH^{1-}	mg/kg	ND	70
carbonate	CO_3^{2-}	mg/kg	87	34
bicarbonate	HCO_3^{1-}	mg/kg	ND	ND
fluoride	F^{1-}	mg/kg	2.5	5.2
chloride	Cl^{1-}	mg/kg	2.0	4.5
sulfate	SO_4^{2-}	mg/kg	17	139
phosphate	PO_4^{3-}	mg/kg	4.7	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	3.2	3.5
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Minimum resistivity per CTM 643, Chlorides per CTM 422, Sulfates per CTM 417

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

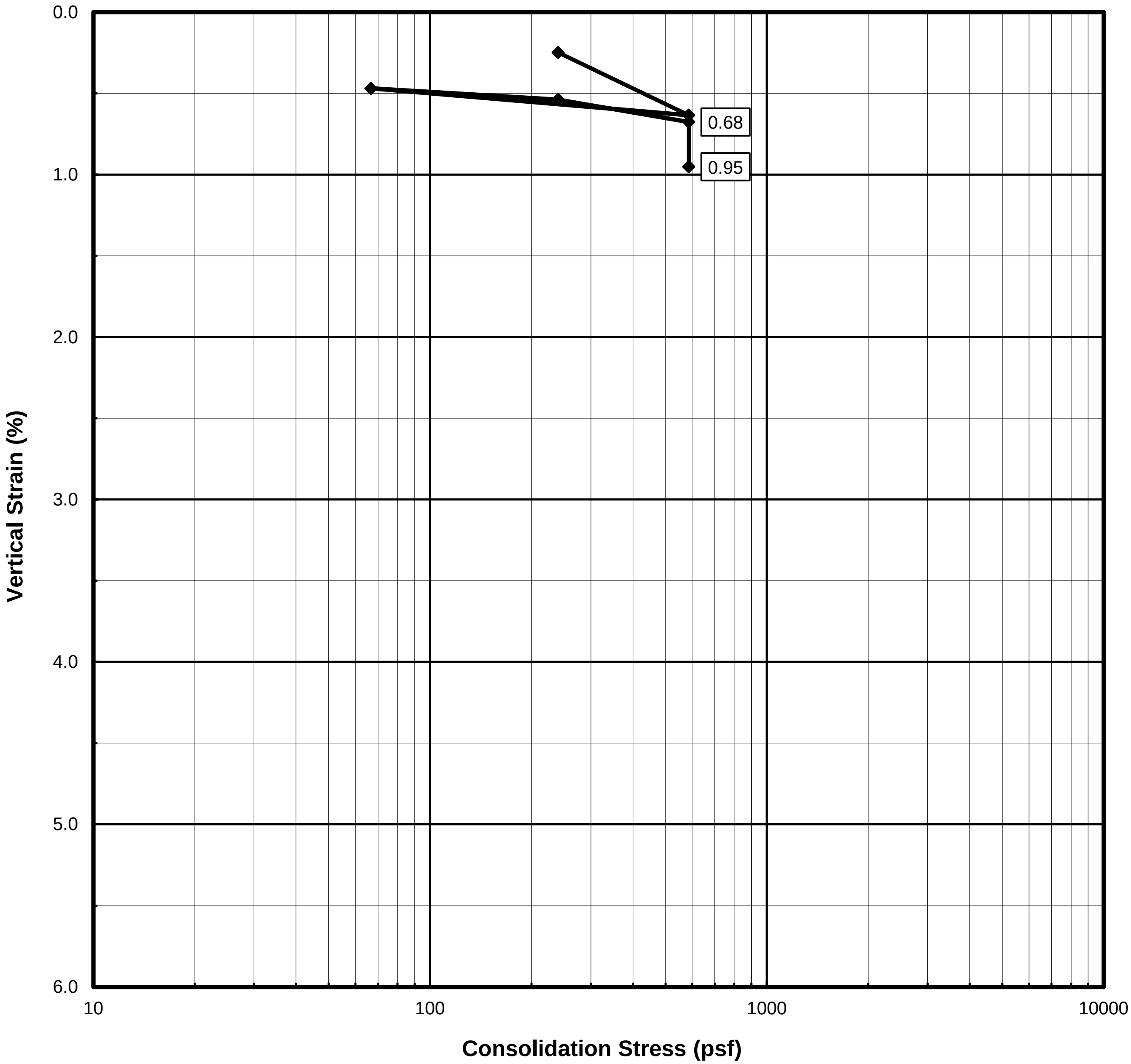
mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316



Description:	**Grayish brown Well Graded SAND with Silt (SW-SM)**										
Boring No.:	B-4		Liquid Limit	---	%			Specific Gravity	2.70	assumed	
Sample No.:	2		Plasticity Index	---	%			Strain for Saturation	28.1	%	
Depth (ft | m)	5.0	1.52	Fines Content	7.7	%			Water added at	586	psf	
								swell strain, ε_s =	-0.3	%	
	Water Content (%)	Total Unit Weight (pcf)	Total Unit Weight (kN/m^3)	Dry Unit Weight (pcf)	Dry Unit Weight (kN/m^3)	Saturation (%)	Void Ratio	Height (inches)	Height (cm)	Diameter (inches)	Diameter (cm)
Initial	2.6	115.8	18.19	112.9	17.74	14.2	0.49	0.725	1.84	2.415	6.134
Final	14.1	127.7	20.06	111.9	17.58	75.6	0.50	0.732	1.86		

URS

MSWD Mission Springs

SWELL / COLLAPSE TEST

ASTM D 4546, Method B

Project Number: 60551186 Date: 10/12/2017 Figure No.:

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

R-VALUE TEST DATA

ASTM D2844

Project Name:	MSWD Mission Springs	Tested By:	ST	Date:	10/09/17
Project Number:	60413030.29865969.00000	Computed By:	KM	Date:	10/11/17
Boring No.:	B-4	Checked By:	AP	Date:	10/12/17

Sample No.: - Depth (ft.): 0-5

Location: N/A

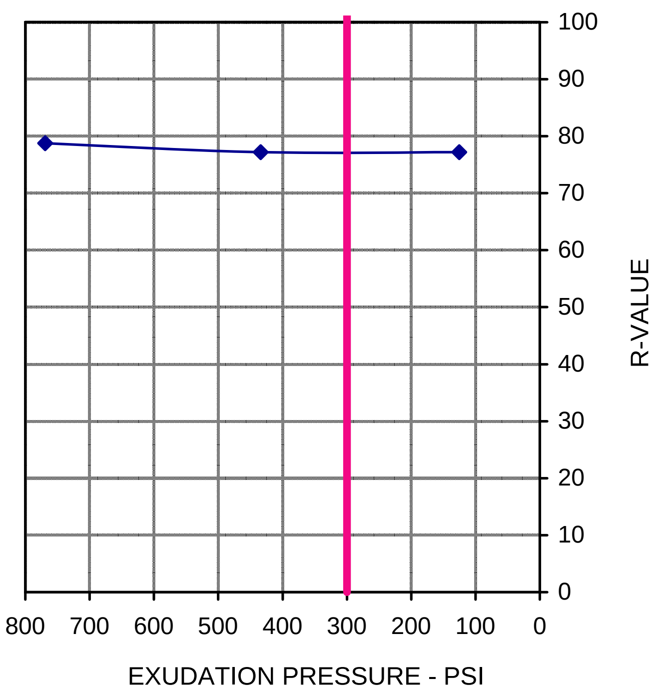
Soil Description: Sand w/silt, fine-coarse grained

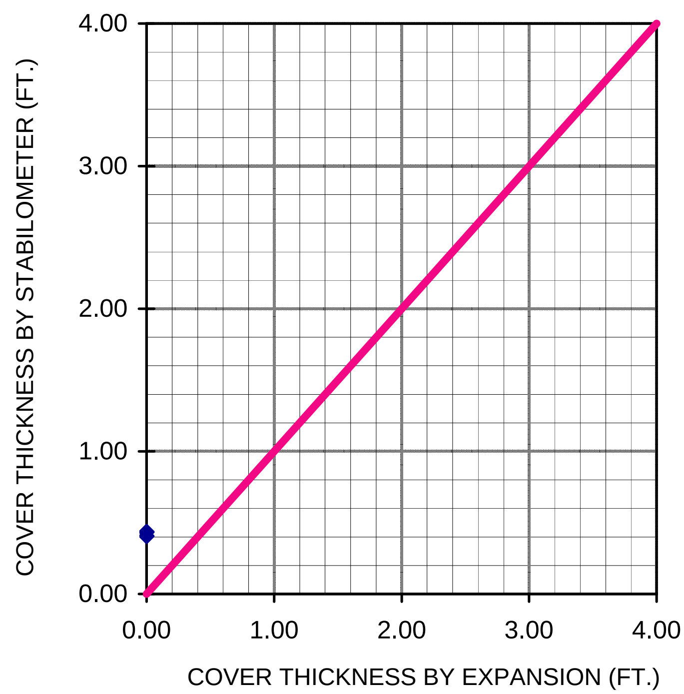
Mold Number	D	E	F	
Water Added, g	71	86	101	
Compact Moisture(%)	7.0	8.4	9.8	
Compaction Gage Pressure, psi	350	250	250	
Exudation Pressure, psi	769	434	125	
Sample Height, Inches	2.7	2.7	2.7	
Gross Weight Mold, g	3131	3127	3045	
Tare Weight Mold, g	1968	1955	1869	
Net Sample Weight, g	1162	1172	1176	
Expansion, inchesx10^{-4}	0	0	0	
Stability 2,000 (160 psi)	11/19	12/20	12/20	
Turns Displacement	5.90	6.05	6.13	
R-Value Uncorrected	76	74	74	
R-Value Corrected	79	77	77	
Dry Density, pcf	121.9	121.3	120.2	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	0.41	0.44	0.44	
G.E. by Expansion	0.00	0.00	0.00	

R-VALUE		
	By Exudation:	77
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	77

Remarks

Gf = 1.34, and 0.0 % Retained on the ¾"
*Not Applicable





Appendix D

Geotechnical Calculations

Design Maps Summary Report

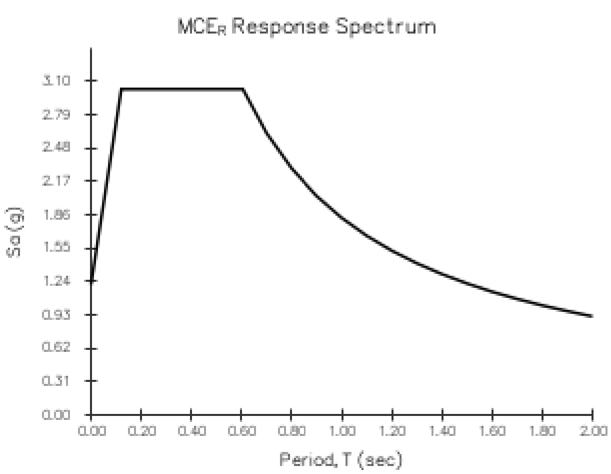
User–Specified Input

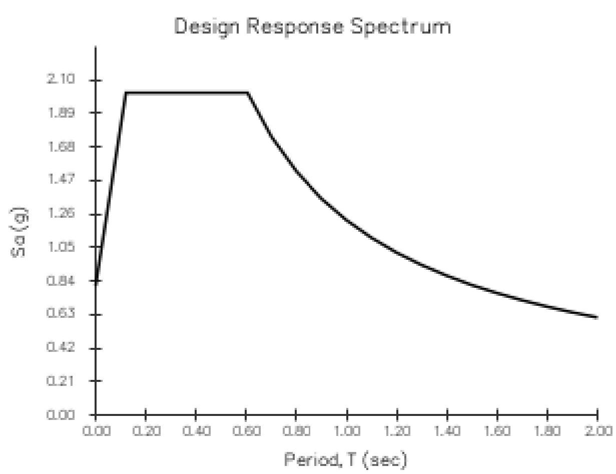
Report Title	Mission Springs Water Treatment Plant Fri October 27, 2017 16:47:22 UTC
Building Code Reference Document	2012/2015 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.90685°N, 116.52902°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	IV (e.g. essential facilities)

USGS–Provided Output

S_S =	3.029 g	S_{MS} =	3.029 g	S_{DS} =	2.020 g
S_1 =	1.222 g	S_{M1} =	1.833 g	S_{D1} =	1.222 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





USGS Design Maps Detailed Report

2012/2015 International Building Code (33.90685°N, 116.52902°W)

Site Class D – "Stiff Soil", Risk Category IV (e.g. essential facilities)

Section 1613.3.1 — Mapped acceleration parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2012/2015 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1] $S_S = 3.029$ g

From Figure 1613.3.1(2) [2] $S_1 = 1.222$ g

Section 1613.3.2 — Site class definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.3.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 3.029 g, F_a = 1.000

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 1.222 g, F_v = 1.500

Equation (16-37):	$S_{MS} = F_aS_S = 1.000 \times 3.029 = 3.029$ g
Equation (16-38):	$S_{M1} = F_vS_1 = 1.500 \times 1.222 = 1.833$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39):	$S_{DS} = ⅔\, S_{MS} = ⅔ \times 3.029 = 2.020$ g
Equation (16-40):	$S_{D1} = ⅔\, S_{M1} = ⅔ \times 1.833 = 1.222$ g

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
S_{DS} < 0.167g	A	A	A
0.167g ≤ S_{DS} < 0.33g	B	B	C
0.33g ≤ S_{DS} < 0.50g	C	C	D
0.50g ≤ S_{DS}	D	D	D

For Risk Category = IV and S_{DS} = 2.020 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
S_{D1} < 0.067g	A	A	A
0.067g ≤ S_{D1} < 0.133g	B	B	C
0.133g ≤ S_{D1} < 0.20g	C	C	D
0.20g ≤ S_{D1}	D	D	D

For Risk Category = IV and S_{D1} = 1.222 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)" = F

Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.3.1(1)*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf
2. *Figure 1613.3.1(2)*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf

Calculation of Subgrade Modulus

Job Name:	Mission Springs Water Treatment Plant		
Job No.	60551186		
Location:	Desert Hot Springs		
Subject:	Subgrade Modulus		
Calculation by:	SD	Checked by:	PY
Date:	10/27/2017		

For Foundations on Sands

B = 40 ft

L = 100 ft

For Dense soils

K(bxb) 180 tcf $\boxed{208.3}$ pci From Figure 6 Navfac manual 7.01

K(LxB) $\boxed{166.7}$ pci

$$k\,(LxB) = \frac{k\,(bxb) * (1+0.5*\,(B/L))}{1.5}$$

Immediate Settlement

$$\Delta hi = \frac{4*q*B^{\wedge}2}{Kv1\ (B+1)\ ^{\wedge}2}$$

From Principles of Foundation Engineering Third Edition
Braja M. Das Eq 4.47 pg 264

q = 1 tsf

AECOM

DATE 10/27/2017
Calculated By: SD
Checked By:

PROJECT: Mission Springs Water Treatment Plant
SUBJECT: Static Settlement Calculations with Consolidation

Data Input:

1. Location A-17-006
2. Depth of GWT 100.0
3. GWT Elevation 200.0
4. Ground Surface Elevation 300.0
5. Bottom of Footing Elevation 298.0

6. Applied Pressure 2.00 ksf
7. Pressure Distribution Boussinesq NA :1 to 3B **NA**

Total Settlement (inch) ***0.6***

Top Layer Elevation (feet)	Top layer depth w.r.t. ground surface (feet)	Bottom layer depth w.r.t. ground surface (feet)	Soil Layer Type	$(N_1)_{60}$	Total unit weight (pcf)	Su (ksf)	Es (ksf)	C'	Cec	Cer	Average layer depth below excavation (feet)	Layer Thickness (feet)	σ_{vo}' (psf)	σ_p' (psf)	$\Delta\sigma_v'$ (psf)	ΔHc_i (inches)
300	0	1	SP	27	115			94		-	0.5	1	58			NA
299	1	2	SP	27	115			94		-	1.5	1	173			NA
298	2	3	SP	27	120			94		-	2.5	1	293		1988	0.114
297	3	4	SP	27	120			94		-	3.5	1	413		1792	0.093
296	4	5	SP	27	120			94		-	4.5	1	533		1481	0.074
295	5	6	SP	27	120			94		-	5.5	1	653		1211	0.058
294	6	7	SP	27	120			94		-	6.5	1	773		1006	0.046
293	7	8	SP	27	120			94		-	7.5	1	893		854	0.037
292	8	9	SP	27	120			94		-	8.5	1	1013		739	0.030
291	9	10	SP	27	120			94		-	9.5	1	1133		649	0.025
290	10	11	SP	50	120			164		-	10.5	1	1253		579	0.012
289	11	12	SP	50	120			164		-	11.5	1	1373		521	0.010
288	12	13	SP	50	120			164		-	12.5	1	1493		474	0.009
287	13	14	SP	50	120			164		-	13.5	1	1613		434	0.008
286	14	15	SP	50	120			164		-	14.5	1	1733		400	0.007
285	15	16	SP	41	120			133		-	15.5	1	1853		371	0.007
284	16	17	SP	41	120			133		-	16.5	1	1973		346	0.006
283	17	18	SP	41	120			133		-	17.5	1	2093		324	0.006
282	18	19	SP	41	120			133		-	18.5	1	2213		304	0.005
281	19	20	SP	41	120			133		-	19.5	1	2333		287	0.005
280	20	21	SP	29	120			99		-	20.5	1	2453		272	0.006
279	21	22	SP	29	120			99		-	21.5	1	2573		258	0.005
278	22	23	SP	29	120			99		-	22.5	1	2693		245	0.005
277	23	24	SP	29	120			99		-	23.5	1	2813		233	0.004
276	24	25	SP	29	120			99		-	24.5	1	2933		223	0.004
275	25	26	SP	27	120			83		-	25.5	1	3053		213	0.004
274	26	27	SP	27	120			83		-	26.5	1	3173		204	0.004
273	27	28	SP	27	120			83		-	27.5	1	3293		196	0.004
272	28	29	SP	27	120			83		-	28.5	1	3413		188	0.003
271	29	30	SP	27	120			83		-	29.5	1	3533		181	0.003
270	30	31	SP	36	120			102		-	30.5	1	3653		174	0.002
269	31	32	SP	36	120			102		-	31.5	1	3773		168	0.002
268	32	33	SP	36	120			102		-	32.5	1	3893		162	0.002
267	33	34	SP	36	120			102		-	33.5	1	4013		156	0.002
266	34	35	SP	36	120			102		-	34.5	1	4133		151	0.002
265	35	36	SP	46	120			125		-	35.5	1	4253		146	0.001
264	36	37	SP	46	120			125		-	36.5	1	4373		141	0.001
263	37	38	SP	46	120			125		-	37.5	1	4493		136	0.001
262	38	39	SP	46	120			125		-	38.5	1	4613		132	0.001
261	39	40	SP	46	120			125		-	39.5	1	4733		128	0.001
260	40	41	SP	35	120			100		-	40.5	1	4853		124	0.001
259	41	42	SP	35	120			100		-	41.5	1	4973		120	0.001
258	42	43	SP	35	120			100		-	42.5	1	5093		117	0.001
257	43	44	SP	35	120			100		-	43.5	1	5213		113	0.001
256	44	45	SP	35	120			100		-	44.5	1	5333		110	0.001
255	45	46	SP	47	120			128		-	45.5	1	5453		107	0.001
254	46	47	SP	47	120			128		-	46.5	1	5573		104	0.001
253	47	48	SP	47	120			128		-	47.5	1	5693		101	0.001
252	48	49	SP	47	120			128		-	48.5	1	5813		99	0.001
251	49	50	SP	47	120			128		-	49.5	1	5933		96	0.001
250	50	51	SP	27	120			83		-	50.5	1	6053		94	0.001
249	51	52	SP	27	120			83		-	51.5	1	6173		91	0.001
248	52	53	SP	27	120			83		-	52.5	1	6293		89	0.001
247	53	54	SP	27	120			83		-	53.5	1	6413		86	0.001
246	54	55	SP	27	120			83		-	54.5	1	6533		85	0.001
245	55	56	SP	27	120			83		-	55.5	1	6653		82	0.001
244	56	57	SP	27	120			83		-	56.5	1	6773		80	0.001
243	57	58	SP	27	120			83		-	57.5	1	6893		79	0.001
242	58	59	SP	27	120			83		-	58.5	1	7013		76	0.001
241	59	60	SP	27	120			83		-	59.5	1	7133		75	0.001
240	60	61	SP	27	120			83		-	60.5	1	7253		73	0.001
239	61	62	SP	27	120			83		-	61.5	1	7373		71	0.001
238	62	63	SP	27	120			83		-	62.5	1	7493		70	0.001
237	63	64	SP	27	120			83		-	63.5	1	7613		68	0.001
236	64	65	SP	27	120			83		-	64.5	1	7733		67	0.001
235	65	66	SP	27	120			83		-	65.5	1	7853		65	0.001
234	66	67	SP	27	120			83		-	66.5	1	7973		63	0.000
233	67	68	SP	27	120			83		-	67.5	1	8093		62	0.000
232	68	69	SP	27	120			83		-	68.5	1	8213		60	0.000
231	69	70	SP	27	120			83		-	69.5	1	8333		60	0.000
230	70	71	SP	27	120			83		-	70.5	1	8453		58	0.000
229	71	72	SP	27	120			83		-	71.5	1	8573		57	0.000
228	72	73	SP	27	120			83		-	72.5	1	8693		56	0.000
227	73	74	SP	27	120			83		-	73.5	1	8813		54	0.000
226	74	75	SP	27	120			83		-	74.5	1	8933		54	0.000
225	75	76	SP	27	120			83		-	75.5	1	9053		52	0.000
224	76	77	SP	27	120			83		-	76.5	1	9173		52	0.000
223	77	78	SP	27	120			83		-	77.5	1	9293		50	0.000
222	78	79	SP	27	120			83		-	78.5	1	9413		49	0.000
221	79	80	SP	27	120			83		-	79.5	1	9533		48	0.000
220	80	81	SP	27	120			83		-	80.5	1	9653		47	0.000
219	81	82	SP	27	120			83		-	81.5	1	9773		47	0.000
218	82	83	SP	27	120			83		-	82.5	1	9893		45	0.000
217	83	84	SP	27	120			83		-	83.5	1	10013		44	0.000
216	84	85	SP	27	120			83		-	84.5	1	10133		44	0.000
215	85	86	SP	27	120			83		-	85.5	1	10253		43	0.000
214	86	87	SP	27	120			83		-	86.5	1	10373		42	0.000
213	87	88	SP	27	120			83		-	87.5	1	10493		41	0.000
212	88	89	SP	27	120			83		-	88.5	1	10613		40	0.000
211	89	90	SP	27	120			83		-	89.5	1	10733		40	0.000
210	90	91	SP	27	120			83		-	90.5	1	10853		39	0.000
209	91	92	SP	27	120			83		-	91.5	1	10973		39	0.000
208	92	93	SP	27	120			83		-	92.5	1	11093		38	0.000
207	93	94	SP	27	120			83		-	93.5	1	11213		37	0.000
206	94	95	SP	27	120			83		-	94.5	1	11333		36	0.000
205	95	96	SP	27	120			83		-	95.5	1	11453		35	0.000
204	96	97	SP	27	120			83		-	96.5	1	11573		35	0.000
203	97	98	SP	27	120			83		-	97.5	1	11693		34	0.000
202	98	99	SP	27	120			83		-	98.5	1	11813		34	0.000
201	99	100	SP	27	120			83		-	99.5	1	11933		33	0.000
200	100	101	SP	27	120			83		-	100.5	1	11990		32	0.000

Total Static Settlement (in) **0.6**

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date	November 6, 2017	
Identification	Spread Footing	

Input

Units of Measurement

	E	SI or E

Foundation Information

Shape	SQ	SQ, CI, CO, or RE
B =	2	ft
L =		ft
D =	1.5	ft

Soil Information

c =	0	lb/ft^2
phi =	32	deg
gamma =	120	lb/ft^3
Dw =	200	ft

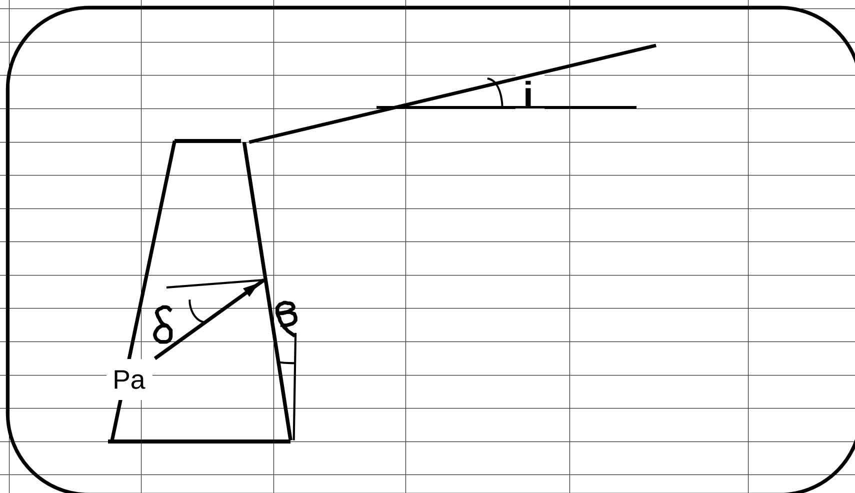
Factor of Safety

F =	3

Results

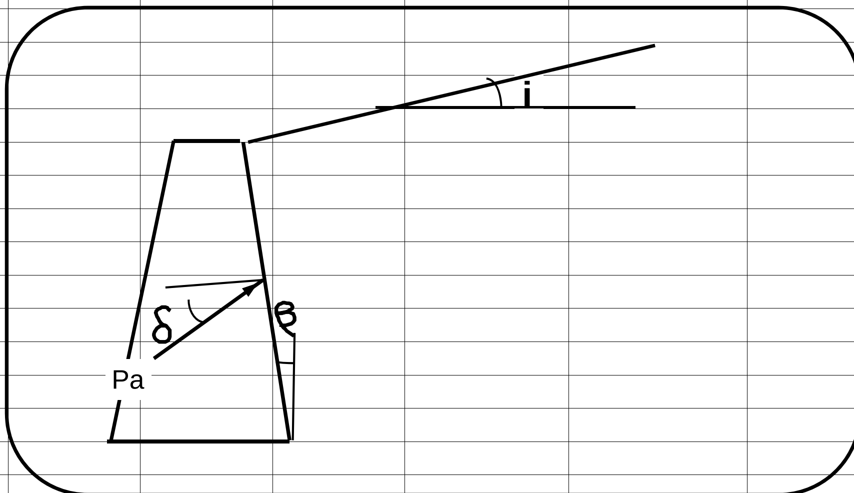
	Terzaghi		Vesic	
Bearing Capacity				
q ult =	7,826	lb/ft^2	10,358	lb/ft^2
q a =	2,609	lb/ft^2	3,453	lb/ft^2
Allowable Column Load				
P =	10	k	14	k

Active Earth Pressures
Coulomb's Theory



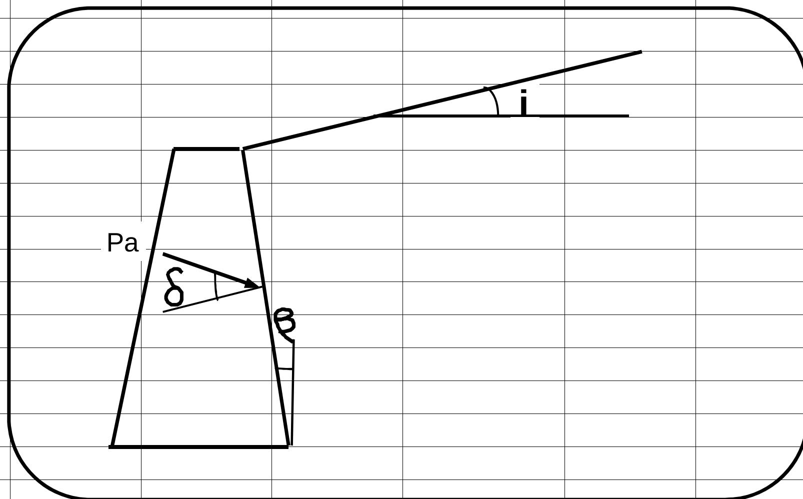
		Degrees	Radians		
β	wall inclination - degrees	0	0.0000000		
δ	wall friction - degrees	0	0.0000000		
i	slope inclination - degrees	0	0.0000000		
ø	soil friction angle - degrees	31	0.5410361		
Ka	Active Earth Pressure Coefficient		0.32		
Kah	Horizontal Component		0.32		
γ_s	Unit Weight of Soil		115		
γ_{ef}	Equivalent Fluid Unit Weight		36.81	Recommend:	37

Active Earth Pressures
Coulomb's Theory



		Degrees	Radians		
β	wall inclination - degrees	0	0.0000000		
δ	wall friction - degrees	0	0.0000000		
i	slope inclination - degrees	0	0.0000000		
ϕ	soil friction angle - degrees	31	0.5410361		
Ka	Active Earth Pressure Coefficient		0.32		
Kah	Horizontal Component		0.32		
γ **s**	Unit Weight of Soil		52.6		
γ **ef**	Equivalent Fluid Unit Weight		16.84	Recommend:	17

Passive Earth Pressures
Coulomb's Theory



		Degrees	Radians	
β	wall inclination - degrees	0	0.0000000	
δ	wall friction - degrees	0	0.0000000	
i	slope inclination - degrees	0	0.0000000	
∅	soil friction angle - degrees	31	0.5410361	
Kp	Passive Earth Pressure Coefficient		3.12	
Kah	Horizontal Component		3.12	
γ_s	Unit Weight of Soil		115	
γ_{ef}	Equivalent Fluid Unit Weight		359	ultimate value

a)	Sand				a)	Sand	
Brace Loads for Internally Braced Flexible Walls					Tie-back excavation		
Total Density		115			Total Density		115
Friction Angle		31			Friction Angle		31
Ka		0.32			Ko		0.48
Uniform xH		24			Uniform xH		25

CALFP Version 1.5

Unit System = E

Title: MSWD RWWTP TI 5 R 50
Traffic Index (TI) = 05.0
R.Value of Subgrade (Native Soil) = 50
Required GE = 0000.80 ft

Base Type = AB-Class 2

Base Gravel Factor = 0001.10
Base R.Value = 0078.00
0.0032*TI*(100-R.VALUE) = 0000.35 ft
Base MAX. depth = 0002.00 ft
Base MIN. depth = 0000.35 ft

Depth (ft)	GF	GE (ft)	Depth (ft)	GF	GE (ft)
00.10	02.54	00.25	00.15	02.54	00.38
00.20	02.54	00.51	00.25	02.54	00.64
00.30	02.54	00.76	00.35	02.54	00.89
00.40	02.54	01.02	00.45	02.54	01.14
00.50	02.54	01.27	00.55	02.56	01.41
00.60	02.64	01.58	00.65	02.71	01.76

HMA Safety Factor (GE) = 0000.20 ft
HMA Ultimate Depth = 0000.65 ft
(HMA MAX. Depth shown in Table)

HMA MIN. Depth (from Base) = 0000.20 ft

HMA MIN. Depth (selected) = 0000.20 ft

Note: Positive Residual GE indicates over-design.
Note: Negative Safety Factor in Base

HMA ft	TPB ft	T-Base ft	B-Base ft	Subbase ft	Res-GE ft	Cost $/y^2	HMA-GF
00.20	00.00	00.35	00.00	00.00	00.09	0000.00	02.54
00.25	00.00	00.35	00.00	00.00	00.22	0000.00	02.54
00.30	00.00	00.35	00.00	00.00	00.35	0000.00	02.54
00.35	00.00	00.35	00.00	00.00	00.47	0000.00	02.54
00.40	00.00	00.35	00.00	00.00	00.60	0000.00	02.54

***** FINISH *****

```
CALFP Version 1.5

Unit System = E

Title:  MSWD RWWTP TI 6 R 50
Traffic Index (TI)  =  06.0
R.Value of Subgrade (Native Soil)  =  50
Required GE  = 0000.96 ft

Base Type   = AB-Class 2

Base Gravel Factor     = 0001.10
Base R.Value           = 0078.00
0.0032*TI*(100-R.VALUE) = 0000.42 ft
Base MAX. depth        = 0002.00 ft
Base MIN. depth        = 0000.35 ft
```

Depth (ft)	GF	GE (ft)	Depth (ft)	GF	GE (ft)
00.10	02.31	00.23	00.15	02.31	00.35
00.20	02.31	00.46	00.25	02.31	00.58
00.30	02.31	00.69	00.35	02.31	00.81
00.40	02.31	00.92	00.45	02.31	01.04
00.50	02.31	01.16	00.55	02.34	01.29
00.60	02.41	01.45	00.65	02.48	01.61
00.70	02.54	01.78	00.75	02.60	01.95
00.80	02.65	02.12	00.85	02.71	02.30

```
HMA Safety Factor (GE)          = 0000.20 ft
HMA Ultimate Depth              = 0000.80 ft
(HMA MAX. Depth shown in Table)

HMA MIN. Depth (from Base)   = 0000.20 ft

HMA MIN. Depth (selected)    = 0000.20 ft


Note:  Positive Residual GE indicates over-design.
Note:  Negative Safety Factor in Base
```

HMA ft	TPB ft	T-Base ft	B-Base ft	Subbase ft	Res-GE ft	Cost $/y^2	HMA-GF
00.25	00.00	00.35	00.00	00.00	00.00	0000.00	02.31
00.30	00.00	00.35	00.00	00.00	00.12	0000.00	02.31
00.35	00.00	00.35	00.00	00.00	00.23	0000.00	02.31
00.40	00.00	00.35	00.00	00.00	00.35	0000.00	02.31
00.45	00.00	00.35	00.00	00.00	00.46	0000.00	02.31
00.50	00.00	00.35	00.00	00.00	00.58	0000.00	02.31

***** FINISH *****

CALFP Version 1.5

Unit System = E

Title: MSWD RWWTP TI 7 R 50
Traffic Index (TI) = 07.0
R.Value of Subgrade (Native Soil) = 50
Required GE = 0001.12 ft

Base Type = AB-Class 2

Base Gravel Factor = 0001.10
Base R.Value = 0078.00
0.0032*TI*(100-R.VALUE) = 0000.49 ft
Base MAX. depth = 0002.00 ft
Base MIN. depth = 0000.35 ft

Depth (ft)	GF	GE (ft)	Depth (ft)	GF	GE (ft)
00.10	02.14	00.21	00.15	02.14	00.32
00.20	02.14	00.43	00.25	02.14	00.54
00.30	02.14	00.64	00.35	02.14	00.75
00.40	02.14	00.86	00.45	02.14	00.96
00.50	02.14	01.07	00.55	02.17	01.19
00.60	02.23	01.34	00.65	02.29	01.49
00.70	02.35	01.65	00.75	02.40	01.80
00.80	02.46	01.97	00.85	02.51	02.13
00.90	02.55	02.30	00.95	02.60	02.47

HMA Safety Factor (GE) = 0000.20 ft
HMA Ultimate Depth = 0000.95 ft
(HMA MAX. Depth shown in Table)

HMA MIN. Depth (from Base) = 0000.20 ft

HMA MIN. Depth (selected) = 0000.20 ft

Note: Positive Residual GE indicates over-design.
Note: Negative Safety Factor in Base

HMA ft	TPB ft	T-Base ft	B-Base ft	Subbase ft	Res-GE ft	Cost $/y^2	HMA-GF
00.35	00.00	00.35	00.00	00.00	00.01	0000.00	02.14
00.40	00.00	00.35	00.00	00.00	00.12	0000.00	02.14
00.45	00.00	00.35	00.00	00.00	00.23	0000.00	02.14
00.50	00.00	00.35	00.00	00.00	00.34	0000.00	02.14
00.55	00.00	00.35	00.00	00.00	00.46	0000.00	02.17
00.60	00.00	00.35	00.00	00.00	00.60	0000.00	02.23

***** FINISH *****

Appendix B

Mission Springs Water District West Valley Water Reclamation Facility PDR Construction Cost Estimate

Specification Division	Item Description	Quantity	Unit	Material Unit Cost	Total Material Cost	Installation Unit Cost	Total Installation Cost	Total Unit Cost	Total	Notes
Division 0-General Conditions										
	Mobilization	1	LS		475,523		437,000	912,524	912,524	5% 0f Division 1-17 Subtotal
	Bonds	1	LS		190,209		174,800	365,009	365,009	2.0% 0f Division 1-17 Subtotal
	Insurance	1	LS		285,314		262,200	547,514	547,514	3.0% 0f Division 1-17 Subtotal
	Overhead and Profit	1	LS		951,047		874,000	1,825,047	1,825,047	10% 0f Division 1-17 Subtotal
	Sales tax	1	LS		760,838		-	760,838	760,838	8% of total material costs
	Miscellaneous Project Expenses	1	LS		80,502		73,980	154,483	154,483	1% of Division 1-17 Subtotal
Subtotal-Division 0					**2,743,434**		**1,821,980**		**4,565,414**	
Division 1- Field Office Requirements										
	Field office	24	MO	-	-	1,500.00	36,000	1,500.00	36,000	
	Temporary storage	24	MO	-	-	1,000.00	24,000	1,000.00	24,000	
	Temporary power	24	MO	150.00	3,600	-	-	150.00	3,600	
	Phone and Internet Service	24	MO	100.00	2,400	-	-	100.00	2,400	
	Project Supervision	24	MO	-	-	24,000.00	576,000	24,000.00	576,000	1 superintendent, 1 PM, 1 / 2 clerical
	Truck	24	MO	1,500.00	36,000	-	-	1,500.00	36,000	
	Computer	24	MO	290.00	6,960	-	-	290.00	6,960	
	Start up and Testing	1	LS	3,000.00	3,000	3,000.00	3,000	6,000.00	6,000	
	Punch list	1	LS	3,000.00	3,000	3,000.00	3,000	6,000.00	6,000	
Subtotal-Division 1									**696,960**	
Division 2- Sitework										
	Site preparation	150,000	SF	-	-	0.25	37,500	0.25	37,500	
	Potholing - Locate Water Connection	10	CY	-	-	35.00	350	35.00	350	
	Underground Utility Connection - Water	1	day	500.00	500	2,500.00	2,500	3,000.00	3,000	
	Excavation IPS	1,500	CY	-	-	15.00	22,500	15.00	22,500	
	Excavation Headworks	800	CY	-	-	15.00	12,000	15.00	12,000	
	Excavation SBR	14,947	CY	-	-	15.00	224,200	15.00	224,200	
	Excavation Decant Tank	4,286	CY	-	-	15.00	64,283	15.00	64,283	
	Excavation MBR	0	CY	-	-	15.00	-	15.00	-	
	Excavation Filters	0	CY	-	-	15.00	-	15.00	-	
	Excavation UV	0	CY	-	-	15.00	-	15.00	-	
	Excavation Cl2 Contact	0	CY	-	-	15.00	-	15.00	-	
	Excavation Sludge Storage	1,871	CY	-	-	15.00	28,062	15.00	28,062	
	Excavation Sludge Thickener Building	190	CY	-	-	15.00	2,846	15.00	2,846	
	Excavation Control Building	600	CY	-	-	15.00	9,000	15.00	9,000	
	Excavation Odor Control	60	CY	-	-	15.00	900	15.00	900	
	Gravel Base Material IPS	50	CY	-	-	25.00	1,250	25.00	1,250	
	Gravel Base Material Headworks	100	CY	60.00	6,000	25.00	2,500	85.00	8,500	
	Gravel Base Material SBR	520	CY	60.00	31,200	25.00	13,000	85.00	44,200	
	Gravel Base Material Decant Tank	200	CY	60.00	12,000	25.00	5,000	85.00	17,000	
	Gravel Base Material MBR	0	CY	60.00	-	25.00	-	85.00	-	
	Gravel Base Material Filters	0	CY	60.00	-	25.00	-	85.00	-	
	Gravel Base Material UV	0	CY	60.00	-	25.00	-	85.00	-	
	Gravel Base Material Cl2 Contact	0	CY	60.00	-	25.00	-	85.00	-	
	Gravel Base Material Sludge Storage	61	CY	60.00	3,680	25.00	1,533	85.00	5,213	
	Gravel Base Material Sludge Thickener Building	117	CY	60.00	7,038	25.00	2,933	85.00	9,971	
	Gravel Base Material Control Building	400	CY	60.00	24,000	25.00	10,000	85.00	34,000	
	Gravel Base Material Odor Control	30	CY	60.00	1,800	25.00	750	85.00	2,550	
	Backfill and Compaction IPS	650	CY	-	-	25.00	16,250	25.00	16,250	
	Backfill and Compaction Headworks	500	CY	-	-	25.00	12,500	25.00	12,500	
	Backfill and Compaction SBR	14,480	CY	-	-	25.00	362,000	25.00	362,000	
	Backfill and Compaction Decant Tank	4,110	CY	-	-	25.00	102,750	25.00	102,750	
	Backfill and Compaction MBR	0	CY	-	-	25.00	-	25.00	-	
	Backfill and Compaction Filters	0	CY	-	-	25.00	-	25.00	-	
	Backfill and Compaction UV	0	CY	-	-	25.00	-	25.00	-	

Specification Division	Item Description	Quantity	Unit	Material Unit Cost	Total Material Cost	Installation Unit Cost	Total Installation Cost	Total Unit Cost	Total	Notes
	Backfill and Compaction Cl2 Contact	0	CY	-	-	25.00	-	25.00	-	
	Backfill and Compaction Sludge Storage	330	CY	-	-	25.00	8,250	25.00	8,250	
	Backfill and Compaction Sludge Thickener Building	0	CY	-	-	25.00	-	25.00	-	
	Backfill and Compaction Control Building	400	CY	-	-	25.00	10,000	25.00	10,000	
	Backfill and Compaction Odor Control	10	CY	-	-	25.00	250	25.00	250	
	Dispose Excess Material IPS	800	CY	60.00	48,000	20.00	16,000	80.00	64,000	Use Excess Spreading Basin Berms
	Dispose Excess Material Headworks	300	CY	60.00	18,000	20.00	6,000	80.00	24,000	Use Excess Spreading Basin Berms
	Dispose Excess Material SBR	470	CY	60.00	28,200	20.00	9,400	80.00	37,600	Use Excess Spreading Basin Berms
	Dispose Excess Material Decant Tank	190	CY	60.00	11,400	20.00	3,800	80.00	15,200	Use Excess Spreading Basin Berms
	Dispose Excess Material MBR	0	CY	60.00	-	20.00	-	80.00	-	Use Excess Spreading Basin Berms
	Dispose Excess Material Filters	0	CY	60.00	-	20.00	-	80.00	-	Use Excess Spreading Basin Berms
	Dispose Excess Material UV	100	CY	60.00	6,000	20.00	2,000	80.00	8,000	Use Excess Spreading Basin Berms
	Dispose Excess Material Cl2 Contact	0	CY	60.00	-	20.00	-	80.00	-	Use Excess Spreading Basin Berms
	Dispose Excess Material Sludge Storage	1,500	CY	60.00	90,000	20.00	30,000	80.00	120,000	Use Excess Spreading Basin Berms
	Dispose Excess Material Sludge Thickener Building	400	CY	60.00	24,000	20.00	8,000	80.00	32,000	Use Excess Spreading Basin Berms
	Dispose Excess Material Control Building	1,100	CY	60.00	66,000	20.00	22,000	80.00	88,000	Use Excess Spreading Basin Berms
	Dispose Excess Material Odor Control	60	CY	60.00	3,600	20.00	1,200	80.00	4,800	Use Excess Spreading Basin Berms
	Fine Grading	75,000	SF	-	-	1.00	75,000	1.00	75,000	
	Loam and Seed	75,000	SF	-	-	0.20	15,000	0.20	15,000	
	AC Pavement	30,000	SF	3.75	112,500	1.88	56,250	5.63	168,750	
	48-inch Manhole and Cover	8	EA	1,600.00	12,800	800.00	6,400	2,400.00	19,200	
	Bollards	24	EA	350.00	8,400	175.00	4,200	525.00	12,600	
Subtotal-Division 2									**1,723,476**	
Division 3- Concrete										
	Reinforced Concrete Cast in Place IPS	200	CY	250.00	50,000	750.00	150,000	1,000.00	200,000	
	Reinforced Concrete Cast in Place Headworks	290	CY	250.00	72,500	750.00	217,500	1,000.00	290,000	
	Reinforced Concrete Cast in Place SBR Tanks	2,378	CY	250.00	594,407	750.00	1,783,222	1,000.00	2,377,630	
	Reinforced Concrete Cast in Place Decant Tanks	625	CY	250.00	156,333	750.00	469,000	1,000.00	625,333	
	Reinforced Concrete Cast in Place Filter Slabs	0	CY	250.00	-	750.00	-	1,000.00	-	
	Reinforced Concrete Cast in Place MBR All	0	CY	250.00	-	750.00	-	1,000.00	-	
	Reinforced Concrete Cast in Place UV	0	CY	250.00	-	750.00	-	1,000.00	-	
	Reinforced Concrete Cast in Place Cl2 Contact	0	CY	250.00	-	750.00	-	1,000.00	-	
	Reinforced Concrete Cast in Place Sludge Storage	410	CY	250.00	102,500	750.00	307,500	1,000.00	410,000	
	Reinforced Concrete Cast in Place Thickener Building	240	CY	250.00	60,000	750.00	180,000	1,000.00	240,000	
	Reinforced Concrete Cast in Place Control Building	1,070	CY	250.00	267,500	750.00	802,500	1,000.00	1,070,000	
	Reinforced Concrete Cast in Place Odor Control	30	CY	250.00	7,500	750.00	22,500	1,000.00	30,000	
Subtotal-Division 3									**5,242,963**	
Division 4- Masonry										
	IPS Walls	880	SF	20.00	17,600	10.00	8,800	30.00	26,400	
	Headworks Walls	2,500	SF	20.00	50,000	10.00	25,000	30.00	75,000	
	Sludge Thickening Building Walls	2,600	SF	20.00	52,000	10.00	26,000	30.00	78,000	
	Control Building Walls	10,800	SF	20.00	216,000	10.00	108,000	30.00	324,000	
Subtotal-Division 4									**503,400**	
Division 5 -Metals										
	Aluminum Grating Headworks	640	SF	40.00	25,600	20.00	12,800	60.00	38,400	
	Aluminum Grating Cl2 Contact	0	SF	40.00	-	20.00	-	60.00	-	
	Aluminum Grating UV	0	SF	40.00	-	20.00	-	60.00	-	
	Sludge Storage Aluminum Tank Covers	3,600	SF	28.00	100,800	14.00	50,400	42.00	151,200	
	Aluminum Hatches IPS	4	EA	6,000	24,000	3,000	12,000	9,000	36,000	
	Aluminum Hatches EQ Tank	8	EA	6,000	48,000	3,000	24,000	9,000	72,000	
	Aluminum Railing	1,200	LF	17.00	20,400	8.50	10,200	25.50	30,600	
	Aluminum Stairs and Railing	5	EA	7,000	35,000	3,500	17,500	10,500	52,500	
	Pipe Supports	1	LS	33,678	33,678	16,839	16,839	50,517	50,517	
Subtotal-Division 5									**431,217**	Allowance

Company Name: AECOM
PDR

MSWD West VAlley WRF
SBR + Sludge Storage
Date:June, 2018

Estimator: RBS
Checker: KWB
CCI: ENR 20 Cities = 11,000.

Specification Division	Item Description	Quantity	Unit	Material Unit Cost	Total Material Cost	Installation Unit Cost	Total Installation Cost	Total Unit Cost	Total	Notes
Division 6 -Wood and Plastics										
	IPS	320	SF	2.00	640	5.00	1,600	7.00	2,240	
	Headworks	1,800	SF	2.00	3,600	5.00	9,000	7.00	12,600	
	Sludge Thickening Building	3,000	SF	2.00	6,000	5.00	15,000	7.00	21,000	
	Control Building	13,600	SF	2.00	27,200	5.00	68,000	7.00	95,200	
Subtotal-Division 6									**131,040**	Allowance
Division 7- Thermal & Moisture Protection										
	Roofing IPS	320	SF	12.00	3,840	6.00	1,920	18.00	5,760	
	Roofing Headworks	1,800	SF	12.00	21,600	6.00	10,800	18.00	32,400	
	Roofing Sludge Thickening Building	3,000	SF	12.00	36,000	6.00	18,000	18.00	54,000	
	Roofing Control Building	13,600	SF	12.00	163,200	6.00	81,600	18.00	244,800	
Subtotal-Division 7									**336,960**	
Division 8- Doors, Windows, & Hardware										
	Doors IPS	1	EA	800.00	800	400.00	400	1,200.00	1,200	
	Doors Headworks	2	EA	800.00	1,600	400.00	800	1,200.00	2,400	
	Doors Sludge Thickening Building	3	EA	800.00	2,400	400.00	1,200	1,200.00	3,600	
	Doors Control Building	12	EA	800.00	9,600	400.00	4,800	1,200.00	14,400	
	Windows IPS	2	EA	400.00	800	200.00	400	600.00	1,200	
	Windows Headworks	2	EA	400.00	800	200.00	400	600.00	1,200	
	Windows Sludge Thickening Building	8	EA	400.00	3,200	200.00	1,600	600.00	4,800	
	Windows Control Building	12	EA	400.00	4,800	200.00	2,400	600.00	7,200	
Subtotal-Division 8									**36,000**	
Division 9- Finishes										
	Painting and Protective Coating	1	LS	112,930	112,930	112,930	112,930	225,860	225,860	
Subtotal-Division 9									**225,860**	Allowance
Division 10- Specialties										
	Signage	1	LS	850.00	850	425.00	425	1,275.00	1,275	
Subtotal-Division 10									**1,275**	Allowance
Division 11- Equipment										
	Influent Pumps	3	EA	$ 50,000	$ 150,000	$ 12,500	$ 37,500	$ 62,500	$ 187,500	
	Influent Pump VFDs	3	EA	$ 8,000	$ 24,000	$ 2,000	$ 6,000	$ 10,000	$ 30,000	
	Coarse Screen	2	EA	$ 129,000	$ 258,000	$ 32,250	$ 64,500	$ 161,250	$ 322,500	
	Grit Removal System	2	EA	$ 80,000	$ 160,000	$ 20,000	$ 40,000	$ 100,000	$ 200,000	
	Grit Pumps	2	EA	$ 25,000	$ 50,000	$ 6,250	$ 12,500	$ 31,250	$ 62,500	
	Grit Dewatering Screw Conveyor	2	EA	$ 60,000	$ 120,000	$ 15,000	$ 30,000	$ 75,000	$ 150,000	
	SBR System	1	LS	$ 1,075,000	$ 1,075,000	$ 268,750	$ 268,750	$ 1,343,750	$ 1,343,750	
	MBR System	0	LS	$ 1,626,000	$ -	$ 406,500	$ -	$ 2,032,500	$ -	
	SBR Effluent EQ Pumps	8	EA	$ 20,000	$ 120,000	$ 5,000	$ 30,000	$ 25,000	$ 150,000	
	SBR Effluent EQ Pump VFDs	8	EA	$ 5,000	$ 30,000	$ 1,250	$ 7,500	$ 6,250	$ 37,500	
	Effluent Filters	0	EA	$ 296,830	$ -	$ 74,208	$ -	$ 371,038	$ -	
	UV System	0	LS	$ 407,000	$ -	$ 101,750	$ -	$ 508,750	$ -	
	Cl2 Pumps	0	LS	$ 15,675	$ -	$ 3,919	$ -	$ 19,594	$ -	
	Sludge Storage Blowers	2	EA	$ 45,000	$ 90,000	$ 11,250	$ 22,500	$ 56,250	$ 112,500	
	Sludge Storage Blower VFD	2	LS	$ 12,000	$ 24,000	$ 3,000	$ 6,000	$ 15,000	$ 30,000	
	Sludge Storage Diffusers	1	LS	$ 30,000	$ 30,000	$ 7,500	$ 7,500	$ 37,500	$ 37,500	
	Sludge Thickening Equipment	1	EA	$ 146,000	$ 146,000	$ 36,500	$ 36,500	$ 182,500	$ 182,500	
	Sludge Thickener Polymer System	1	EA	$ 8,000	$ 8,000	$ 2,000	$ 2,000	$ 10,000	$ 10,000	
	Sludge Thickener Grinders	1	EA	$ 30,000	$ 30,000	$ 7,500	$ 7,500	$ 37,500	$ 37,500	
	Sludge Thickener Feed Pumps	1	EA	$ 30,000	$ 30,000	$ 7,500	$ 7,500	$ 37,500	$ 37,500	
	Sludge Thickener Feed Pump VFD	1	EA	$ 6,000	$ 6,000	$ 1,500	$ 1,500	$ 7,500	$ 7,500	
	Thickened Sludge Pumps	1	EA	$ 30,000	$ 30,000	$ 7,500	$ 7,500	$ 37,500	$ 37,500	
	Thickened Sludge Pump VFDs	1	EA	$ 6,000	$ 6,000	$ 1,500	$ 1,500	$ 7,500	$ 7,500	
	Sludge Dewatering Equipment	2	EA	$ 420,000	$ 840,000	$ 105,000	$ 210,000	$ 525,000	$ 1,050,000	

MSWD West VAlley WRF
SBR + Sludge Storage
Date:June, 2018

Estimator: RBS
Checker: KWB
CCI: ENR 20 Cities = 11,000.

Specification Division	Item Description	Quantity	Unit	Material Unit Cost	Total Material Cost	Installation Unit Cost	Total Installation Cost	Total Unit Cost	Total	Notes
	Sludge Dewatering Polymer System	2	EA	$ 25,000	$ 50,000	$ 6,250	$ 12,500	$ 31,250	$ 62,500	
	Sludge Dewatering Grinders	2	EA	$ 30,000	$ 60,000	$ 7,500	$ 15,000	$ 37,500	$ 75,000	
	Sludge Dewatering Feed Pumps	2	EA	$ 26,000	$ 52,000	$ 6,500	$ 13,000	$ 32,500	$ 65,000	
	Sludge Dewatering Feed Pump VFD	2	EA	$ 9,000	$ 18,000	$ 2,250	$ 4,500	$ 11,250	$ 22,500	
	Sludge Dewatering Conveyors	2	EA	$ 125,000	$ 250,000	$ 31,250	$ 62,500	$ 156,250	$ 312,500	
	Dewatered Sludge Storage Bin	1	EA	$ 120,000	$ 120,000	$ 30,000	$ 30,000	$ 150,000	$ 150,000	
	Dewatered Sludge Weigh Scale	1	EA	$ 6,000	$ 6,000	$ 1,500	$ 1,500	$ 7,500	$ 7,500	
	Central Odor Control Unit and Fan	1	LS	$ 172,000	$ 172,000	$ 43,000	$ 43,000	$ 215,000	$ 215,000	
Subtotal-Division 11									**4,943,750**	
Division12- Furnishings										
	Work Station	2	EA	1,100.00	2,200	550.00	1,100	1,650.00	3,300	Allowance
Subtotal-Division 12									**3,300**	Allowance
Division 13- Special Construction										
	Not Used	-		-	-	-	-	-	-	
Subtotal-Division 13									**-**	
Division 14- Conveying Systems										
	Jib Cranes	8	EA	3,000	24,000	1,500.00	12,000	4,500	36,000	
Subtotal-Division 14									**36,000**	Allowance
Division 15- Mechanical										
	Yard Piping	1	LS	494,375.00	494,375	247,188	247,188	741,563	741,563	
	SBR Process Piping	1	LS	134,375	134,375	67,188	67,188	201,563	201,563	
	MBR Process Piping	0	LS	-	-	-	-	-	-	
	Filter Process Piping	0	LS	-	-	-	-	-	-	
	UV Process Piping	0	LS	-	-	-	-	-	-	
	Cl 2 Contact Process Piping	0	LS	-	-	-	-	-	-	
	IPS Process Piping	1	LS	21,750	21,750	10,875	10,875	32,625	32,625	
	IPS Plumbing	1	LS	4,350	4,350	2,175	2,175	6,525	6,525	
	IPS HVAC	1	LS	8,700	8,700	4,350	4,350	13,050	13,050	
	IPS Odor Control Duct	1	LS	9,788	9,788	4,894	4,894	14,681	14,681	
	Headworks Process Piping	1	LS	73,500	73,500	36,750	36,750	110,250	110,250	
	Headworks Plumbing	1	LS	14,700	14,700	7,350	7,350	22,050	22,050	
	Headworks HVAC	1	LS	29,400	29,400	14,700	14,700	44,100	44,100	
	Headworks Odor Control Duct	1	LS	33,075	33,075	16,538	16,538	49,613	49,613	
	Thickener Building Process Piping	1	LS	32,000	32,000	16,000	16,000	48,000	48,000	
	Thickener Building Plumbing	1	LS	6,400	6,400	3,200	3,200	9,600	9,600	
	Thickener Building HVAC	1	LS	12,800	12,800	6,400	6,400	19,200	19,200	
	Thickener Building Odor Control Duct	1	LS	14,400	14,400	7,200	7,200	21,600	21,600	
	Control Building Process Piping	1	LS	174,500	174,500	87,250	87,250	261,750	261,750	
	Control Building Plumbing	1	LS	34,900	34,900	17,450	17,450	52,350	52,350	
	Control Building HVAC	1	LS	69,800	69,800	34,900	34,900	104,700	104,700	
	Control Building Odor Control Duct	1	LS	78,525	78,525	39,263	39,263	117,788	117,788	
Subtotal-Division 15									**1,871,006**	Allowance
Division 16- Electrical										
	Standby Power	200	KW	500.00	100,000	250	50,000	750	150,000	
	Power Service	1	LS	30,000	30,000	15,000	15,000	45,000	45,000	
	IPS Electrical Power and Control	1	LS	30,450	30,450	15,225	15,225	45,675	45,675	
	IPS Electrical Lighting	1	LS	17,400	17,400	8,700	8,700	26,100	26,100	
	Headworks Electrical Power and Control	1	LS	102,900	102,900	51,450	51,450	154,350	154,350	
	Headworks Electrical Lighting	1	LS	58,800	58,800	29,400	29,400	88,200	88,200	
	SBR Power and Control Electrical	1	LS	188,125	188,125	94,063	94,063	282,188	282,188	
	MBR Power and Control Electrical	0	LS	-	-	-	-	-	-	
	Filter Power and Control Electrical	0	LS	-	-	-	-	-	-	
	UV Power and Control Electrical	0	LS	-	-	-	-	-	-	

Specification Division	Item Description	Quantity	Unit	Material Unit Cost	Total Material Cost	Installation Unit Cost	Total Installation Cost	Total Unit Cost	Total	Notes
	Cl 2 Contact Power and Control Electrical	0	LS	-	-	-	-	-	-	
	Thickener Building Electrical Power and Control	1	LS	44,800	44,800	22,400	22,400	67,200	67,200	
	Thickener Building Electrical Lighting	1	LS	25,600	25,600	12,800	12,800	38,400	38,400	
	Control Building Electrical Power and Control	1	LS	244,300	244,300	122,150	122,150	366,450	366,450	
	Control Building Electrical Lighting	1	LS	139,600	139,600	69,800	69,800	209,400	209,400	
	Odor Control Electrical Power and Control	1	LS	30,100	30,100	15,050	15,050	45,150	45,150	
Subtotal-Division 16									**1,518,113**	Allowance
Division 17- Instrumentation										
	IPS Instrumentation	1	LS	10,875	10,875	5,438	5,438	16,313	16,313	
	IPS SCADA	1	LS	6,525	6,525	3,263	3,263	9,788	9,788	
	Headworks Instrumentation	1	LS	36,750	36,750	18,375	18,375	55,125	55,125	
	Headworks SCADA	1	LS	22,050	22,050	11,025	11,025	33,075	33,075	
	SBR Instrumentation	1	LS	67,188	67,188	33,594	33,594	100,781	100,781	
	SBR SCADA	1	LS	40,313	40,313	20,156	20,156	60,469	60,469	
	MBR Instrumentation	0	LS	-	-	-	-	-	-	
	MBR SCADA	0	LS	-	-	-	-	-	-	
	Filter Instrumentation	0	LS	-	-	-	-	-	-	
	Filter SCADA	0	LS	-	-	-	-	-	-	
	UV Instrumentation	0	LS	-	-	-	-	-	-	
	UV SCADA	0	LS	-	-	-	-	-	-	
	Cl 2 Contact Instrumentation	0	LS	-	-	-	-	-	-	
	Cl 2 Contact SCADA	0	LS	-	-	-	-	-	-	
	Thickener Building Instrumentation	1	LS	16,000	16,000	8,000	8,000	24,000	24,000	
	Thickener Building SCADA	1	LS	9,600	9,600	4,800	4,800	14,400	14,400	
	Control Building Instrumentation	1	LS	87,250	87,250	43,625	43,625	130,875	130,875	
	Control Building SCADA	1	LS	52,350	52,350	26,175	26,175	78,525	78,525	
	Odor Control Instrumentation	1	LS	10,750	10,750	5,375	5,375	16,125	16,125	
	Odor Control SCADA	1	LS	6,450	6,450	3,225	3,225	9,675	9,675	
Subtotal-Division 17									**549,150**	Allowance
Subtotal-Divisions 1-17					9,510,470		8,740,000		18,250,470	
Division 0					2,743,434		1,821,980		4,565,414	
SUBTOTAL					**12,253,903**		**10,561,981**		**22,815,884**	
CONTINGENCY (20%)					2,450,781		2,112,396		4,563,177	
TOTAL ESTIMATED CONSTRUCTION COST					**14,704,684**		**12,674,377**		**27,379,061**	
DESIGN ENGINEERING (+/- 6.5%)					796,504		686,529		1,483,032	
CONSTRUCTION ENGINEERING (+/- 10.0%)					1,225,390		1,056,198		2,281,588	
PROGRAM AND ADMINISTRATIVE COSTS					245,078		211,240		456,318	
TOTAL 2018 ESTIMATED PROJECT COST					**16,971,656**		**14,628,343**		**31,600,000**	
NOTES:										
1. ENR 20 Cities INDEX = 11,000 (June, 2018)										
2.Contractors Insurance, Bonds, Profit & Overhead included in Division 0 for this Cost Estimate										
3 Project Services includes design engineering, construction engineering, commissioning support, legal-fiscal expenses, and contingency										