LiSWA WWTRF Phase 1 Improvement Project

Basis of Design Report

Prepared for:
City of Lincoln

Prepared by:
Stantec Consulting Services Inc.

November 22, 2024

Sign-off Sheet

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1.0 PURPOSE AND SCOPE

The purpose of this Basis of Design Report (BODR) is to provide the Lincoln-SMD1 Wastewater Authority (LiSWA) with the basic design concepts for the WWTRF Phase 1 Improvements Project. This report includes the design criteria, process features, and discipline-specific code requirements for the project.

2.0 FLOWS AND LOADS

Table 1 summarizes the projected flows and loads for average dry weather flow (ADWF) of 6, 7.1, 8 Mgal/d. The proposed project is aimed at designing the secondary process for 6 Mgal/d (ADWF) and annual average loads and the rest of unit processes to have capacity to meet the peak flows associated with the 8 Mgal/d ADWF. The last column of **Table 1** summarizes the flows and loads for this project.

Table 1 Design Flows and Loads

		ADWF			
Parameter	Unit	6 Mgal/d	7.1 Mgal/d (1)	8 Mgal/d (2)	New Design Criteria
Flow					
ADWF	Mgal/d	6.0	7.1	8.0	6.0
PMF	Mgal/d	15.0	17.0	18.4	18.4
PDF	Mgal/d	27.0	30.5	32.8	32.8
PHF	Mgal/d	40.8	46.2	49.6	49.6
BOD Loads					
AAL	lb/day	16,513	19,541	22,018	16,513
PML	lb/day	20,642	24,426	27,522	20,642
PDL	lb/day	33,026	39,081	44,035	33,026
TSS Loads					
AAL	lb/day	16,513	19,541	22,018	16,513
PML	lb/day	20,642	24,426	27,522	20,642
PDL	lb/day	33,026	39,081	44,035	33,026
TKN Loads					
AAL	lb/day	3,204	3,791	4,271	3,204
PML	lb/day	4,004	4,739	5,339	4,004
PDL	lb/day	6,407	7,582	8,543	6,407
Peak Flow Factors					
PMF/ADWF		2.5	2.4	2.3	3.1
PDF/ADWF		4.5	4.3	4.1	5.5
PHF/ADWF		6.8	6.5	6.2	8.3
Peak Load Factors					
PML/AAL		1.25	1.25	1.25	1.25
PDL/AAL		2.00	2.00	2.00	2.00

(1) 7.1 Mgal/d ADWF is achieved with the addition of Oxidation Ditch No. 4.
(2) 8.0 Mgal/d ADWF is achieved with the addition of Oxidation Ditch No. 4 and Secondary Clarifier No. 4.

3.0 INFLUENT PUMP STATION

The influent pump station has space for a total of six pumps. Existing facilities include five large pumps, each rated at 5,500 gpm, and one small pump rated at 2,250 gpm. With one large pump out of service, the reliable pump station capacity is 34.8 Mgal/d. As shown in **Table 1**and **Table 2**, the estimated peak hour influent flow is 49.6 Mgal/d for the proposed project.

Therefore, it is recommended to replace all existing pumps with six submersible pumps each with a capacity of 6,945 gpm (10 Mgal/d), resulting in a total reliable capacity of 50 Mgal/d for this project.

Table 2 Influent Pump Station Design Criteria

Parameter	Unit	Existing Conditions	New Design Criteria
Peak Hour Flow	Mgal/d	29.5	49.6[(a)]
Reliable Pump Capacity	Mgal/d	34.8	50
Small Pumps			
Number	Each	1	
Motor Power	HP	35	
Capacity	GPM	2250	
TDH	ft	46	
Pump Type	Each	Submersible Pump	
Model	Each		
Large Pumps			
Number	Each	5	6
Motor Power	HP	85	125
Capacity	GPM	5,500	6,945
TDH	ft	47	49
Pump Type	Each	Submersible Pump	Submersible Pump
Model	Each	Xylem/Flygt model NP3301-624LT	Xylem/Flygt model NP-3356.716

(a) including in-plant recycle

4.0 INFLUENT SCREENS

There are no changes to the influent screening within the Phase 1 Improvements Project. There are two existing automatic screens and a bypass screen. The automatic screens include a screenings washer compactor. Each screen has approximately 22 Mgal/d of capacity. To convey the required 49.6 Mgal/d with two screens, the channel freeboard is reduced to less than 2 feet, and/or the bypass screen channel can also be online for added screening capacity.

5.0 GRIT REMOVAL

The original headworks design includes provisions for adding two forced-vortex-type grit removal basins downstream the two mechanical screens. However, since redundancy is not critical for grit removal, one larger grit removal basin is recommended to reduce the project cost. This project will include the installation of one 50 Mgal/d grit removal basin. The design criteria of the grit removal system are shown in **Table 3**. The location of the grit removal system is between the influent screens and the Parshall flow meter.

Table 3 Grit Removal Design Criteria

Parameter	Unit	Existing Conditions	New Design Criteria
Peak Hour Flow	Mgal/d	29.5	50
New Grit Basins			
Number	Each		1
Type	--		Vortex
Capacity	Mgal/d		50
Peak Removal Rate, 50 Mesh & Larger	%		95
Grit Basin Propeller Drive	HP		2
Grit Basin Drive	HP		5
Grit Removal Pump	HP		25
Grit Pump Capacity	GPM		500

6.0 SECONDARY TREATMENT

This project targets wastewater flows and loads at 6 Mgal/d ADWF. The flow and load capacity are higher than the original plant design of 5.9 Mgal/d. The plant capacity increase can be achieved without building new basins or clarifiers by lowering the Sludge retention time and reducing the peak flow allowed to secondary treatment. A side-by-side design criteria is shown in **Table 4**. The additional capacity is achieved by diverting peak flow to the Emergency Storage Basin and allowing limited solids wash out of the secondary clarifiers under critical conditions.

Table 4 Secondary Treatment Design Criteria

Parameter	Unit	Original Design Criteria	This Project Design Criteria
Secondary Influent Flows and Loads			
ADWF	Mgal/d	5.9	6.0
Max Allowable Flow	Mgal/d	29.5	23.6
BOD Loads			
AAL	lb/day	14,000	16,513
PML	lb/day	18,200	20,642
PDL	lb/day	25,300	33,026
TSS Loads			
AAL	lb/day	14,000	16,513
PML	lb/day	18,200	20,642
PDL	lb/day	25,300	33,026
TKN Loads			
AAL	lb/day	3,200	3,204
PML	lb/day	3,900	4,004
PDL	lb/day	5,600	6,407
Process Design			
Min. Temp	C	15	16
Total SRT	days	16	13.5
Oxidation Ditches			
Number	Each	3	3
Volume (Each)	Mgal	3.12	3.12
Secondary Clarifiers			
Number	Each	3	3
Diameter	ft	110	110
RAS Pump Station #1			
Number of RAS Pumps	Each	3	3
Capacity (Each)	gpm	3,800	3,800
RAS Pump Station #2			
Number of RAS Pumps	Each	2	2
Capacity (Each)	gpm	3,800	3,800

7.0 MATURATION PONDS PUMP STATION

The maturation pond pump station has space for five mixed flow pumps, which are currently filled with five identical pumps, providing a reliable capacity (with one pump out of service) of 35.1 Mgal/d. Based on the peak hour flows shown in **Table 5** this capacity is not adequate for a target 8 Mgal/d ADWF. All five pumps will be replaced to attain a total reliable capacity of 50.4 Mgal/d, which is adequate for the 8.0 Mgal/d ADWF plant.

Table 5 Maturation Ponds Pump Station Design Criteria

Parameter	Unit	Existing Conditions	New Design Criteria
Peak Hour Flow	Mgal/d	29.5	49.6[(a)]
Reliable Pump Capacity	Mgal/d	35.1	50.4
Small Pumps			
Number	Each	5	
Motor Power	HP	60	
Capacity	GPM	6100	
TDH	ft	22.1	
Pump Type		Vertical Turbine Pump	
Model		Flowserve 16 DH 60-6 D PROP 60 hz	
Large Pumps			
Number	Each		5
Motor Power	HP		100
Capacity	GPM		8,754
TDH	ft		23.30
Pump Type	Each		Vertical Turbine Pump
Model	Each		Flowserve 18AFV-DH, 23.5 ° Vane Angle

(a) including in-plant recycle

8.0 MATURATION PONDS

There are no changes to the existing maturation ponds. The Maturation ponds provide priority pollutant equalization and peak flow attenuation (equalization) to the tertiary plant (DAF, filters and UV facilities). The maturation ponds consist of two basins providing a total volume of 173 million gallons.

9.0 MATURATION POND EFFLUENT PUMP STATION

When the maturation ponds have a high water level, water can be directed to the tertiary portion of the plant by gravity. Effluent from the maturation ponds discharge through two existing maturation pond outlet structures before reaching the maturation pond level control structure, where it is then diverted to the DAF system. When levels in the ponds are too low for gravity flow, two existing submersible pumps within the outlet structures are used to convey additional flow. These pumps each have a capacity of 4.0 Mgal/d, which is much less than the design peak month flow required (plus plant recycle flows) of 20.6 Mgal/d, as shown in **Table 6**.

In addition to increased pumping capacity additional storage volume is also required in the ponds. To increase the available volume required for equalization in the maturation ponds the minimum pond water level needs to be lowered. This new minimum water level will be elevation 101.3 feet, which is lower than the existing outlet weir elevation of 109.1 feet which allows gravity flow to the DAF system. Therefore, at low water levels a new effluent pump station is required to convey peak month flow and recycle flow to the tertiary facilities. The pump station will increase pumping capacity to tertiary facilities and allow all of the available equalization volume to be utilized.

The new Maturation Pond Effluent Pump Station includes three new same pumps, which will result in total of five pumps with a reliable capacity of about 19.32 Mgal/d. This is slightly less than the target flow rate of 20.6 Mgal/d, but this limitation only exists when the maturation ponds are at their minimum water level. Target flows can be achieved and exceeded at all other water levels. It was determined that 19.32 Mgal/d at minimum pond water levels is acceptable because the selected pump model exists at multiple locations around the existing facility and matching this equipment is desirable.

Table 6 Maturation Pond Effluent Pump Station Design Criteria

Parameter	Unit	Existing Conditions	New Design Criteria
Equalized Peak Month Flow from Mat Ponds	Mgal/d	11.9	20.6
Reliable Pump Capacity	Mgal/d	4	19.32
Low Water Level	ft		101.3
Maximum Surface Level	ft	114	114
Pumps			
Number	Each	2	5
Motor Power	HP	25	25
Capacity	GPM	2,780	3,550
TDH	ft		16
Pump Type	Each	Submersible Pump	Submersible Pump
Model	Each	Xylem/Flygt NP3171-614LT	Xylem/Flygt NP3171-614LT

10.0 DISSOLVED AIR FLOATATION SYSTEM

There are two existing dissolved air floatation (DAF) clarifiers with ancillary facilities to remove algae from the maturation pond. Each DAF unit has a capacity of 8.0 Mgal/d. No DAF expansion is included with this project. Although the existing reliable capacity (with one DAF out of service) can only generate 8 Mgal/d, the total capacity of 16 Mgal/d. This is still less than the peak flows from the Maturation Pond Effluent Pump Station, but this is mitigated by having less algae during the winter when peak flows typically occur, the DAFS can be flooded and convey additional flow and still perform acceptably, and the DAF can be bypassed. See **Table 7**.

Table 7 Dissolved Air Floatation Design Criteria

Parameter	Unit	Existing Conditions
Equalized Peak Month Flow from Mat Ponds	Mgal/d	20.6
Total Capacity	Mgal/d	16.0
Reliable Capacity	Mgal/d	8.0
DAF Units	Each	2
Recirculation Pumps		
Type	-	Vertical Turbine
Number	Each	3
Capacity	gpm	1300
Horsepower	HP	75
Float Pumps		
Type	-	Progressive Cavity
Number	Each	2
Capacity	gpm	135
Horsepower	HP	15

11.0 FILTER FEED PUMP STATION

The filter feed pump station has spaces for five mixed flow pumps but four are currently installed: two large and two small pumps, with a reliable capacity of 15.9 Mgal/d. Since peak plant influent flows are equalized in the maturation ponds, the new design peak flow for the filter feed pumps is 20.6 Mgal/d, which is equal to peak month flows plus plant recycle flow.

It is recommended to replace two existing small pumps with two large pumps and add one additional large pump, which will result in total of five large pumps with a reliable capacity of about 28.5 Mgal/d. This capacity exceeds the required flow rate, but the condition of both small pumps warrants replacement.

Table 8 Filter Feed Pump Station Design Criteria

Parameter	Unit	Existing Conditions	New Design Criteria
Peak Month Flow + Recycle	Mgal/d	11.9	20.6
Reliable Pump Capacity	Mgal/d	14.4	28.5
Small Pumps			
Number	Each	2	
Motor Power	HP	25	
Capacity	GPM	2,524.5	
TDH	ft	20.2	
Pump Type		Vertical Turbine Pump	
Model		Flowserve 16 DH 25-6 D PROP 60 hz	
Large Pumps			
Number	Each	2	5
Motor Power	HP	60	60
Capacity	GPM	5,950	4,950
TDH	ft	23.2	29.20
Pump Type	Each	Vertical Turbine Pump	Vertical Turbine Pump
Model	Each	One (1) Flowserve 15AFV-DH, 22° Vane Angle One (1) Flowserve 16 DH 60-6 D PROP 60 hz Note: Different name but same performance	Four (4) Flowserve 15AFV-DH, 22° Vane Angle One (1) Flowserve 16 DH 60-6 D PROP 60 hz Note: Different name but same performance

12.0 FILTERS

The existing filter system was laid out to accommodate six filter cells on both sides of a common mudwell (12 cells total). Only six filter cells on one side of the mudwell are existing, and each filter cell has a surface area of 384 square feet. Therefore, the reliable filter area (one cell out of service) is 1,920 square feet. Using a maximum loading rate of 5 gpm/ft^2, the maximum allowable filter influent flow is 13.8 Mgal/d. As shown in **Table 9**. This project will expand the filters to 18.4 Mal/d plus 12% in-plant recycle (20.6 Mgal/d total).

Although Eight filter cells (seven duty cells and one standby) can only generate a reliable capacity of 19.4 Mgal/d, the total capacity of 22.6 Mgal/d can accommodate the peak month flow with all eight cells in operation, shown in **Table 9**. In addition to filter cells, this project will install one rapid mixing basin and two flocculation basins.

Table 9 Effluent Filtration Design Criteria

Parameter	Unit	Existing Conditions	New Design Criteria
Peak Month Flow + 12%	Mgal/d	11.9	20.6
Reliable Pump Capacity	Mgal/d	13.8	19.4
Total Capacity	Mgal/d	16.6	22.6
Maximum Loading Rate	GPM/sqft	5.0	5.0
Filter			
Type	-	Sand, Pulsed Bed	Sand, Pulsed Bed
Number of Cells	Each	6	8
Cell Dimension	ft x ft	32 x 12	32 x 12
Filter Area per Cell	sqft	384	384
Total cell surface area	sqft	1920	3072
Rapid Mixing			
Number of Mixers/Basins	Each	1	2
Horsepower	HP	3	3
Volume	Gal	1,940	1,940
Detention Time, Peak Month	Sec	20	20
Velocity Gradient "G"	1/Sec	610	610
Flocculation			
Type of Mixers/Basins	-	Vertical Shaft	Vertical Shaft
Number of Flocs Basins	Each	2	4
Horsepower	HP	1	1
Total Basin Volume	Gal	83,000	83,000
Detention Time, Peak Month	Min	17	17
Velocity Gradient 'G', 1st Stage	1/Sec	90	90
Velocity Gradient 'G', 2nd Stage	1/Sec	50	50

13.0 UV DISINFECTION

The existing UV disinfection system is comprised of six channels with five of them equipped to meet current disinfection targets. The system has a current design capacity of 17.5 Mgal/d based on delivering a minimum UV dose of 100 mJ/cm^2 at a design minimum UV transmittance (UVT) of 70%.

This project upgrades and expands the UV system with to 20.6 Mgal/d with the newest version of the Wedeco (a Xylem brand) TAK55 system, with an in-channel cleaning system and control equipment. All six UV channels will receive new UV equipment (banks, modules, lamps, quartz sleeves, pneumatically driven automatic wiping systems, ballasts and ballast enclosures, instrumentation, junction boxes, etc.) Additionally, a new control cabinet with redundant Allen Bradley ControlLogix programmable logic controllers (PLCs) will be provided to improve operation reliability and flexibility. A summary of the UV disinfection system design criteria for the project is shown in **Table 10**.

Table 10 UV Disinfection System Expansion Design Criteria

Design Criteria	Value
Manufacturer / Model	Wedeco / TAK55 H (110 mm lamp centerline spacing) (1)
Peak Month Flow + In-Plant Recycle Flows	20.6 Mgal/d
UV Disinfection System Design Peak Flow Capacity	3.6 Mgal/d per channel (21.6 Mgal/d total)
Design Minimum UV Dose	100 mJ/cm^2
Design Minimum UV Transmittance (UVT)	70% @ 254 nm
Channels	6 (6 duty)
Banks per Channel	5 (4 duty, 1 standby)
Modules per Bank	3
Lamp Type	Low Pressure High Output
Lamps per Module	12
Lamps per Channel	180 (144 duty, 36 standby)
Total Number of Lamps in System	1,080 (864 duty, 216 standby)
Design End of Lamp Life (EOLL) Value	0.87 (guaranteed lamp life of 14,000 hours) (2)
Design Fouling Factor (FF) Value	0.80
Effluent Finger Weir Length / Top Elevation	720 inches (60 feet, total perimeter) / 107.81 feet (3)
Required Channel Width	25 13/16 inches (4)
Effluent Total Coliform Permit Requirements	< 2.2 MPN/100 mL (7-day median) < 23 MPN/100 mL (cannot exceed more than once in any 30-day period) < 240 MPN/100 mL (at all times)

(1) Based on the January 2010 validation report by Carollo Engineers titled Wedeco Open Channel TAK-55 Wastewater UV Reactor 320W Validation Report, which meets the requirements of the Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse (National Water Research Institute in collaboration with Water Research Foundation, August 2012, Third Edition).
(2) Ecoray ELR-30 lamps have a third party validated end of lamp life (EOLL) of 0.87 for 14,000 hours of operation. Stantec has contacted the Division of Drinking Water (DDW) to request approval to use a design EOLL of 0.87. The peak flow capacity presented in this table assumes that DDW will approve using a design EOLL of 0.87.
(3) The effluent finger weirs are required to be replaced to increase the weir length and lower the top of weir elevation. Wedeco provided a preliminary total weir length and top of weir elevation. The final values shall be confirmed by Wedeco.
(4) The TAK55 system with the 110 mm lamp centerline spacing has a required channel width of 25 13/16 inches. The width of the existing channels (currently 28 inches) will be reduced using 304 stainless steel plates on both sides of the channel (to protect the coating on the channel walls). Refer to drawings for additional information.

14.0 EFFLUENT PUMP STATION

The effluent pump station has space for a total of five pumps. Existing facilities include two small pumps, both rated as 3,600 gpm, and one large pump rated as 4,700 gpm, and an extra-large pump rated as 6,000 gpm. With the extra-large pump out of service, the reliable pump station capacity is 20.4 Mgal/d. As shown in **Table 11**, the permit discharge limit for Auburn Ravine Creek is 25 Mgal/d.

Therefore, it is recommended to replace the existing two small pumps and one extra-large pump with four large pumps rated as 4,810 gpm. resulting in a total reliable capacity of 25 Mgal/d for this project.

Table 11 Effluent Pump Station Design Criteria

Parameter	Unit	Existing Conditions	New Design Criteria
Permit Discharge Limit	Mgal/d	25	25
Reliable Pump Capacity	Mgal/d	20.4	25.0
Small Pumps			
Number	Each	2	
Motor Power	HP	30	
Capacity	GPM	3,600	
TDH	ft	25	
Pump Type	Each	Submersible Pump	
Model	Each	Xylem/Flygt model CP3201-821	
Large Pumps			
Number	Each	1	1 (existing)
Motor Power	HP	60	60
Capacity	GPM	4,700	4,700
TDH	ft	38	38
Pump Type	Each	Submersible Pump	Submersible Pump
Model	Each	Xylem/Flygt model CP3300-804LT	Xylem/Flygt model CP3300-804LT
Extra-Large Pumps			
Number	Each	1	4 (new)
Motor Power	HP	60	60
Capacity	GPM	6,000	4,810
TDH	ft	31	39
Pump Type	Each	Submersible Pump	Submersible Pump
Model	Each	Xylem/Flygt model NP3301-814LT	Xylem/Flygt model NP3202-614

(a) 25 Mgal/d criteria can only be achieved when Auburn Ravine is not in a flood stage and the discharge is flowing over an unsubmerged outfall weir. During a flood stage in Auburn Ravine, all five pumps will be needed to discharge flow to the outfall, or the excess flow (beyond 23 Mgal/d) will need to be diverted

15.0 EFFLUENT STORAGE, REUSE, AND DISPOSAL FACILITIES

There are no changes to the effluent, reuse or disposal facilities included with the proposed project. There are 190 million gallons of storage in the existing Tertiary Storage Basins 1 and 2. The Reclamation Booster Pump Station has a reliable capacity of 6.3 Mgal/d, depending on the discharge location, and the facility has approximately 900 acres of reclamation land onsite and contractually off-site with the Machado Farm and the City of Lincoln.

16.0 SOLIDS TREATMENT AND HANDLING

With no expansion to solids treatment or dewatering, it is expected that some solids dewatering may be required on weekends with the proposed project. The design does not include a second solids storage tank for this project, and depending on actual plant performance, it may be determined that weekend dewatering operations can be avoided.

17.0 GEOTECHNICAL DESIGN

Blackburn Consulting (BCI) performed three (3) geotechnical design reports (Nov. 2017, Feb. 2018 and Apr. 2018) and presented design recommendations for Lincoln WWTRF expansion project, as documented in the appendix. Two geotechnical update letters were provided June 4, 2024 and are also in the appendix.

18.0 STRUCTURAL DESIGN

Design of structures, structural components and equipment anchorages will comply with the design codes, standards, and project references listed below:

- Design shall conform to the 2022 current edition of the California Building Code.
- Loading criteria and loading combinations for buildings and structures shall conform to the current edition of the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7) and ASCE 7 Supplements.
- Design and placement of structural concrete shall conform to the current edition of the American Concrete Institute Building Code Requirements for Reinforced Concrete (ACI 318).
- Design and placement of concrete for liquid containment structures shall follow the current edition of the American Concrete Institute Code Requirements for Environmental Engineering Concrete Structures (ACI 350) in addition to the requirements of ACI 318.

- Design, fabrication, and erection of structural steel shall follow the current edition of the AISC Manual of Steel Construction.

19.0 ELECTRICAL DESIGN

The electrical system shall be designed to support the additional facility improvements at the WWTRF as presented in this report. The plant's existing electrical distribution system was designed to facilitate planned future upgrades and, where feasible, existing switchboard and motor control center (MCC) spares or space will be used to serve the added loads.

The expected electrical improvements required for this project include a new motor control center (MCC) sized for the Phase 1 loads. The MCC will connect to a spare switch at the existing pad mounted switchgear PSW-202A. The existing plant's Main Switchgear will require an upgrade of the existing medium voltage fuse size feeding PSW-202A to accommodate the added loads.

The existing 2000 kW/2500 kVA, 12.47 kV rated generator does not have sufficient capacity for the proposed electrical loads. Additional emergency generator capacity and load shedding schemes will be required. The design will include a permanently installed generator connected to MCC-100 through a new automatic transfer switch (ATS). The ATS will replace the existing manual keyed interlock circuit breakers and portable generator connector to allow immediate transfer of power between the utility and generator. The generator and ATS will be sized for existing and future loads connected to MCC-100.

Because of the planned design principles and the use of advanced control elements in the existing plant design, it will be possible to specify equipment and components that are nearly identical to the existing equipment to maintain plant standardization.

20.0 INSTRUMENTATION AND CONTROL

The new facilities will integrate into the existing SCADA system, with additional Allen Bradley PLCs as needed. SCADA modifications will ensure balanced loading of the emergency power system, continuing the existing WWTRF concepts in this project.

21.0 SITE PAVING AND GRADING

Site grading will ensure proper stormwater drainage and capture of spills. Paved access will be provided for operational needs, with subgrade preparation to ensure stability. All buildings will be situated above the 100-year flood plain elevation, continuing the existing WWTRF concepts in this project. Most improvements will be implemented within the footprint of existing facilities and do not require paving or grading improvements.

22.0 STORM DRAINAGE

Stormwater will be managed through existing conveyance systems and stored in the Stormwater Detention Basin (SDB). The system is designed to handle specified storm events and ensure controlled discharge to Orchard Creek, continuing to the existing WWTRF concepts in this project.

23.0 YARD PIPING

Piping will maintain flow requirements with appropriate slopes and materials. The drainage network will include cleanouts and manholes for maintenance, continuing the existing WWTRF concepts in this project. Process piping will primarily be an extension of the existing piping strategy between discrete unit processes, much of which was already oversized and will accommodate the proposed project.

APPENDIX A

Lincoln WWTRF Review of Maturation Pond and Tertiary Storage Operation and Sizing and Impacts on Other Facilities Based on Updated Data and New Permit Temperature Requirements, by Stantec, April 2023

Lincoln WWTRF Review of Maturation Pond and Tertiary Storage Operation and Sizing and Impacts on Other Facilities Based on Updated Data and New Permit Temperature Requirements

April 13, 2023

Prepared for:

Sewer Maintenance District No. 1 Wastewater Authority (LiSWA)

Prepared by:

Stantec Consulting Services Inc.

LINCOLN WWTRF REVIEW OF MATURATION POND AND TERTIARY STORAGE OPERATION AND SIZING AND IMPACTS ON OTHER FACILITIES BASED ON UPDATED DATA AND NEW PERMIT TEMPERATURE REQUIREMENTS

Revision	Description	Author		Quality Check		Independent Review	

LINCOLN WWTRF REVIEW OF MATURATION POND AND TERTIARY STORAGE OPERATION AND SIZING AND IMPACTS ON OTHER FACILITIES BASED ON UPDATED DATA AND NEW PERMIT TEMPERATURE REQUIREMENTS

This document entitled Lincoln WWTRF Review of Maturation Pond and Tertiary Storage Operation and Sizing and Impacts on Other Facilities Based on Updated Data and New Permit Temperature Requirements was prepared by Stantec Consulting Services Inc. (“Stantec”) for the account of LiSWA (the “Client”). Any reliance on this document by any third party is strictly prohibited. The material in it reflects Stantec’s professional judgment in light of the scope, schedule and other limitations stated in the document and in the contract between Stantec and the Client. The opinions in the document are based on conditions and information existing at the time the document was published and do not take into account any subsequent changes. In preparing the document, Stantec did not verify information supplied to it by others. Any use which a third party makes of this document is the responsibility of such third party. Such third party agrees that Stantec shall not be responsible for costs or damages of any kind, if any, suffered by it or any other third party as a result of decisions made or actions taken based on this document.

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Jeffrey R Hauser

April 13, 2023

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Gabe Aronow

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1.0 INTRODUCTION, PURPOSE, AND BACKGROUND

The Basis of Design Report for the City of Lincoln Wastewater Treatment and Reclamation Facility (WWTRF) Phase 1 and Phase 2 Expansion Project by Stantec, dated August 24, 2017, hereinafter referred to as the 2017 BODR, recommended major modifications to the maturation pond facilities and expansion of the tertiary storage basins. Recent heavy rainfalls and high plant flows necessitate re-evaluation of maturation pond operations and sizing, while revised effluent temperature limits listed below necessitate re-evaluation of tertiary storage requirements.

Effluent temperature limits for the Lincoln WWTRF are currently being revised pursuant to a site-specific study in Auburn Ravine Creek. Key requirements expected to be adopted are generally as follows:

> The discharge shall not cause the annual average receiving stream temperature to increase more than 5 °F compared to the ambient stream temperature and shall not cause the receiving stream temperature to rise above:
>
> a. 68 °F on a 7-day average of daily maximums basis from 1 October through 31 December
>
> b. 64 °F on a 7-day average of daily maximums basis from 1 January through 31 May
>
> c. 5 °F over the ambient background temperature as a daily average for the period from 1 June through 30 September
>
> d. 5 °F over the ambient background temperature as a daily average if ambient receiving background temperatures meet or exceed 68 °F or 64 °F per a and b, respectively.

These temperature limits will govern when and how much discharge can be made to Auburn Ravine Creek. Effluent that cannot be discharged to Auburn Ravine Creek based on the temperature limits or used for irrigation must be stored in the tertiary storage basins at the WWTRF.

1.1 PURPOSE OF THIS STUDY

The purpose of this study is to re-evaluate the recommended designs of the maturation ponds, tertiary storage basins, and other facilities impacted by the design and/or operation of the maturation ponds and tertiary storage basins based on recent data and new permit requirements.

1.2 BACKGROUND FOR MATURATION PONDS

The 2017 BODR recommended modifications to the maturation pond facilities were based on historical wastewater flows and rainfall records from mid-2004 to mid-2012, transformed to represent future conditions when the average dry weather flow (ADWF) increases to 8.0 Mgal/d. This was an update of the analysis previously prepared for the Midwestern Placer Regional Sewer Project Preliminary Design Report, dated November 20, 2012, hereinafter referred to as the 2012 PDR. The rainfall records considered included an approximate 12-year return frequency 30-day total rainfall of 13.49 inches in January 2006; however, conditions occurring in March 2011 with a 30-day rainfall total of 9.89 inches were more severe for determining maturation pond equalization storage requirements. Based on a design peak month average tertiary treatment capacity of 15.3 Mgal/d, a maturation pond equalization volume of 51 Mgal was determined. To obtain this useful volume, the minimum water level in the maturation ponds would have to be reduced to a water surface elevation of 107.7 ft (later revised to 108.5 ft), which is below the existing minimum outlet weir elevation (109.1 ft). The high flow requirement and the new low level in the maturation ponds resulted in the need for a new Maturation Pond Effluent Pump Station. Although the minimum maturation pond storage requirement for flow equalization was 51 Mgal at a minimum water surface elevation of 107.7 ft (later revised to 108.5 ft), the Maturation Pond Effluent Pump Station was designed (but not yet built) to provide additional flexibility to allow pumping the design flow rate of 15.3 Mgal/d at a maturation pond water level as low as 105.8 ft, providing for a minimum residual volume (minimum pool) of about 96 Mgal, a minimum hydraulic retention time of about 6.3 days (average for the two maturation ponds), and a useable equalization storage volume (above minimum pool) of about 81 Mgal.

Maturation pond storage volumes versus water surface elevation are shown in Figure 1-1.

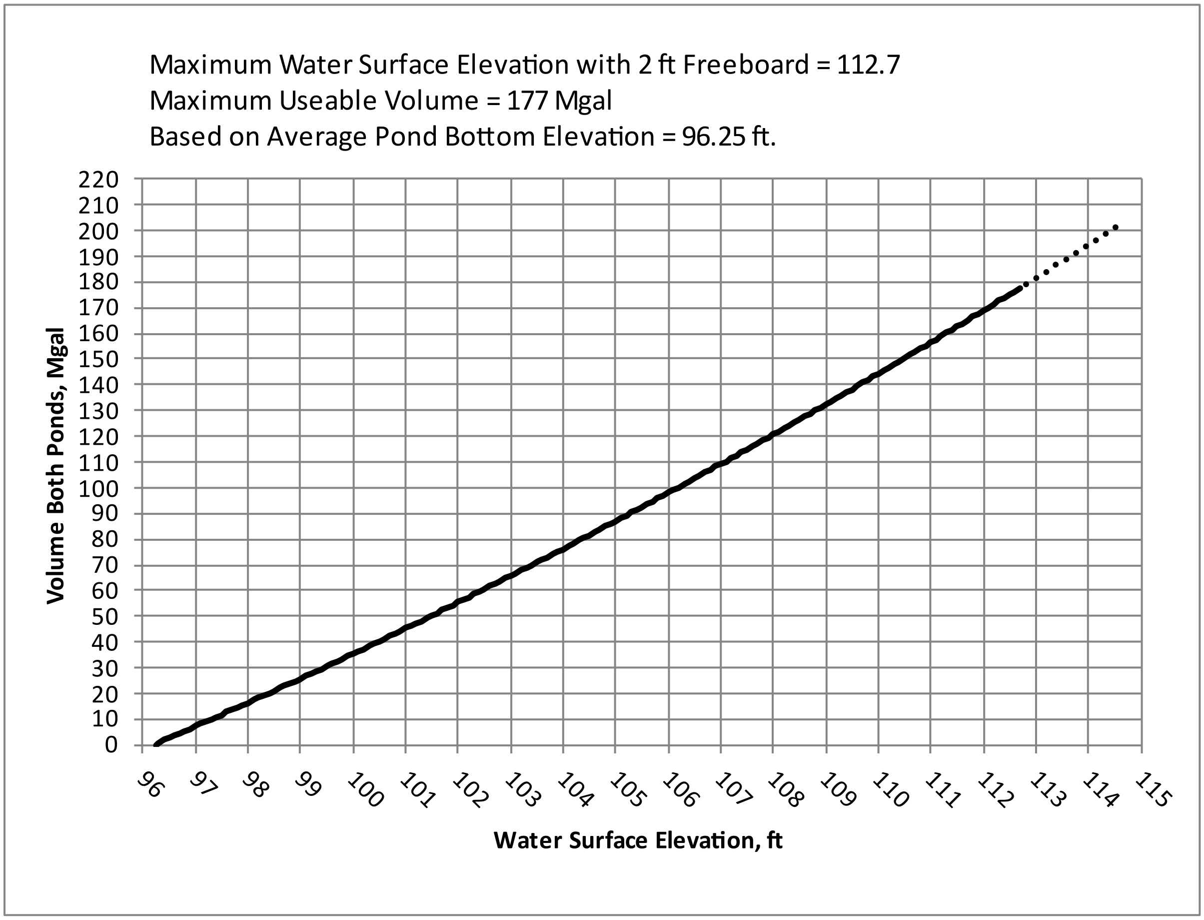

Figure 1-1 Maturation Pond Storage Volume vs Water Surface Elevation

1.3 BACKGROUND FOR TERTIARY STORAGE BASINS

At the time of the 2017 BODR, temperature provisions a, b, and d listed above were not included in the discharge permit and were not part of the analysis. The applicable discharge permit at that time required that the discharge shall not cause the temperature in the receiving stream to increase more than 5 °F over the ambient background temperature at any time.

The 2017 BODR describes the analysis of daily data from the beginning of 2005 through June 2017 on wastewater effluent and Auburn Ravine Creek flows and temperatures to determine what the allowable discharge would have been on each day based on the then-current temperature limits and based on overriding maximum allowable discharges of 12.2 Mgal/d (the then-current permit limit) and 20.4 Mgal/d. From the analysis, Water Year 2014 (October 2013 through September 2014) was selected as the year with the most restrictive allowable discharges in the months of October through March when storage would typically be required under the previous temperature requirements. The monthly average allowable discharges determined for Water Year 2014 were used as input to a water balance model to determine the amount of effluent stored each month and the maximum accumulated storage volume for the year. Plant influent flows used in the water balance were flows projected to occur when the average dry weather flow (ADWF) reaches 8.0 Mgal/d. From the water balance calculations, it was determined that the amount of tertiary storage required would be 270 Mgal and 232 Mgal, based on the overriding maximum discharges of 12.2 and 20.4 Mgal/d, respectively. Both results are based on having 942 acres (the current area) available for irrigation reuse.

The 2017 BODR also included evaluation of 100-year return frequency rainfall conditions to determine if the higher wastewater flows and higher rainfall accumulations in plant facilities would result in more stringent tertiary storage requirements than the Water Year 2014 analysis. Because of higher creek flows and higher allowable discharges in 100-year rainfall conditions, tertiary storage requirements were less than those determined for Water Year 2014.

For the 2017 BODR analysis and for actual plant operations until mid-2018, Auburn Ravine Creek temperatures upstream of the Lincoln discharge (at monitoring station “R1” or “R3”, which have been used interchangeably) were based on daily grab determinations, usually made at around 8:30 am. This is important because creek temperatures later in the day would typically be higher. Using the lower temperature at 8:30 am results in more restrictive discharge limits when the objective is to avoid a temperature increase of more than 5 °F. This is because the colder creek water would be impacted more severely by warmer wastewater effluent.

2.0 UPDATED EVALUATION OF MATURATION PONDS

As originally conceived, the maturation ponds were designed to provide two main functions: 1) dilution (by blending) and incidental removals to reduce peak concentrations of priority pollutants, and 2) flow equalization to allow downstream facilities to be designed for the average maturation pond effluent flow during peak month flow conditions. Incidental benefits of the maturation ponds are that they provide for substantial cooling of the wastewater flow prior to creek discharge, which is helpful in meeting permitted temperature impacts to the creek, they provide natural disinfection, making it much easier to comply with effluent coliform limits after ultraviolet (UV) disinfection, and they provide an additional barrier for removal of suspended solids ahead of the filters in the event of a secondary treatment process overload or upset.

The dilution of priority pollutants was investigated in the 2012 PDR and reviewed for the 2017 BODR. Actual performance data for the maturation ponds indicate statistically significant reductions in average concentrations of priority pollutants. In addition to the dilution effect, reductions in concentrations also could be due to other factors, such as biological, chemical, and physical transformations. Without extensive studies and frequent monitoring of actual concentrations of various priority pollutants entering, within, and exiting the maturation ponds over a long period of time and including all seasons of the year, it is not possible to evaluate the actual impacts on pollutant concentrations and how those impacts would vary with differing pond volumes. Recognizing that significant priority pollutant dilution should occur with hydraulic retention times of at least 5 days, even if not specifically quantified, a minimum hydraulic retention time of 5 days was incorporated in the 2017 BODR.

It should be noted that the priority pollutant dilution benefits of the maturation ponds are based on diluting short-term spikes of pollutant concentrations. For example, if the maturation ponds hydraulic retention time is 5 days and a priority pollutant concentration spike occurs on one day, that spike is diluted into the maturation pond contents that reflect the effects of the previous four days (and more) without the pollutant. The actual reduction in pollutant concentration obtained by dilution will depend on mixing characteristics in the ponds and other factors. If a plant influent pollutant concentration is sustained over many days, there would be little, if any, dilution impact in the maturation ponds.

The potential "spikey" nature of influent priority pollutant concentrations means that spike events would likely go unnoticed, because priority pollutant monitoring occurs only once per year. Similarly, the benefits of the maturation ponds in reducing such pollutant concentrations, even if substantial, would also go unnoticed. This is particularly true because only the plant effluent (after the maturation ponds) is monitored for priority pollutants, so there are no available before and after data being routinely monitored and recorded.

Considering the above, there are legitimate questions regarding the cost/benefit ratio of the maturation pond priority pollutant concentration reduction function.

In this study, the possibility of bypassing most flows around the maturation ponds is considered for wet season operations, recognizing this would eliminate most of the potential benefit of priority pollutant concentration reduction, while considering that such reductions may not be necessary for compliance with priority pollutant regulations (California Toxics Rule). Unfortunately, bypassing most flows around the maturation pond would result in loss of the incidental benefits mentioned above (cooling, disinfection, and secondary process backup) and would result in other issues, which are discussed later in this document.

The equalization storage function of the maturation ponds is accomplished by varying the water level in the ponds, while not allowing the level to drop below the minimum water level desired for priority pollutant dilution (as applicable) or other operational considerations. The equalization volume must be adequate to 1) accumulate excess peak wet weather flows that exceed the capacity of the downstream tertiary treatment facilities, and 2) provide for desired diurnal flow equalization for the tertiary treatment system. The volume required for the first objective is far greater than that for the second.

2.1 MATURATION POND OPERATIONAL CONCEPTS

Two concepts for maturation pond operation are considered in this study as shown in Figure 2-1.

The mainstream configuration represents existing operations. In this case, all of the secondary effluent is routed through the maturation ponds. Accordingly, the Maturation Pond Feed Pump Station must be sized to handle the design peak hour flow that could be routed through the secondary process, which includes an allowance for in-plant recycle streams and for rainfall collected on the plant site and processed through the plant. Rainfall capture on the plant site is new based on the facility stormwater permit and the desire to minimize sampling, analysis and assicated stormwater monitoring costs. As indicated in the 2017 BODR for the 8 Mgal/d design condition, the design capacity for the pump station would be 36.5 Mgal/d. However, based on recent peak flow data, this capacity should be increased to perhaps 50.0 Mgal/d (to be determined - see footnote (a) under Table 5-1 later in this document). The Maturation Pond Effluent Pump Station must be sized for the design maximum equalized peak flow to the downstream facilities, which include the dissolved air flotation (DAF) system, filters, UV disinfection system, and subsequent facilities. In the 2017 BODR, a design capacity of 15.3 Mgal/d was indicated. However, as developed later in this section, this capacity may need to be increased based on recent peak wet weather flow data.

In the mainstream configuration the minimum pool volume available for priority pollutant dilution is determined as the maximum pond volume minus the volume needed for flow equalization. In the 2017 BODR, the minimum volume needed for equalization was indicated to be 51 Mgal; however, as previously indicated pumping flexibility was provided during design to allow this to increase to 81 Mgal, leaving 96 Mgal available for priority pollutant dilution. At the peak maturation pond effluent design flow of 15.3 Mgal/d, the minimum hydraulic retention time would be 6.3 days (average for both ponds). However, the volume needed for equalization and the volume available for priority pollutant dilution must now be reviewed based on the same recent peak wet weather flows mentioned above.

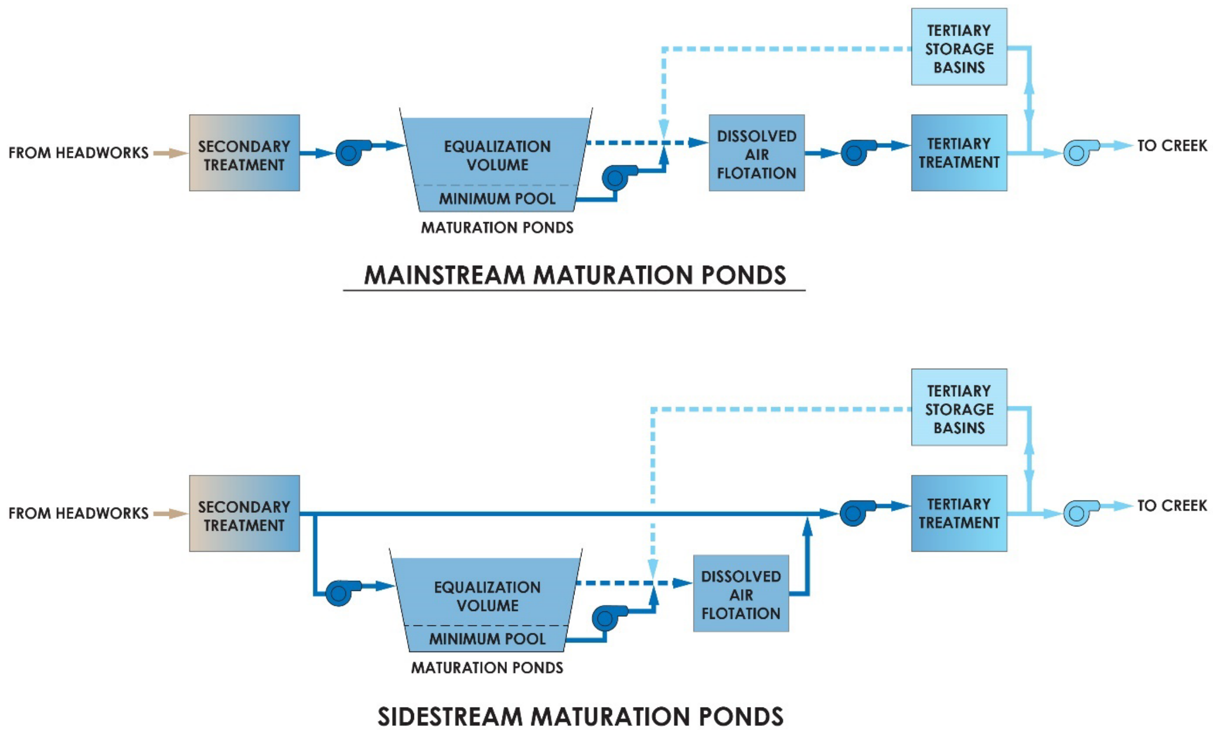

Figure 2-1 Maturation Pond Operations Concepts

In the sidestream configuration, the tertiary treatment equalized flow, which would include most of the secondary effluent, would be routed directly to the filters. Secondary effluent flows greater than the tertiary treatment equalized flow would be pumped to the maturation ponds. In this case, the Maturation Pond Feed Pump Station capacity would be much lower than the 50.0 Mgal/d (to be verified) capacity needed for the mainstream concept. For example, if the design peak tertiary treatment flow was 20 Mgal/d, the Maturation Pond Feed Pump Station would be required to handle 50.0-20.0 = 30.0 Mgal/d. The required capacity is considered later in this document.

Similarly, in the sidestream configuration, the required capacity of the Maturation Pond Effluent Pump Station would be much less than that for the mainstream configuration. For the sidestream arrangement, the capacity would be determined based on the difference between the minimum secondary effluent flow (i.e., the lowest flow occurring during the day) and the desired flow to the tertiary treatment system during a maturation pond drawdown operation. To maximize the drawdown rate and empty the maturation pond equalization storage volume as soon as possible after a peak flow event, the flow to the tertiary treatment system would be the design peak flow for this system. Again, using a hypothetical example, if the design peak tertiary treatment flow was 20 Mgal/d and the minimum secondary effluent flow during maturation pond drawdown was say 7 Mgal/d, the required Maturation Pond Effluent Pump Station flow would be 20-7=13 Mgal/d. However, it may not be necessary to accomplish drawdown as fast as possible, in which case the pump capacity could be reduced. This topic is addressed later in this section.

With sidestream maturation ponds, providing an equalized flow to the DAF system, filters, and downstream facilities becomes much more complex than with mainstream maturation ponds. With the mainstream scenario, the DAF and filter flow simply would be the controlled outflow from the maturation ponds. With the sidestream scenario, equalized flow to the DAF and filters would require coordinated diversions to the maturation ponds when secondary effluent flow exceeds the desired filter flow and returns from the maturation ponds when secondary effluent flow is less than the desired filter flow. Therefore, four flow rates must be monitored and controlled in a coordinated manner (secondary effluent flow, filter inflow, maturation pond inflow, and maturation pond outflow). Three pump stations would be involved in the control scheme: Filter Feed Pump Station, Maturation Pond Feed Pump Station, and Maturation Pond Effluent Pump Station. Furthermore, recognizing that the DAF system cannot be turned on and off to allow sporadic returns from the maturation ponds, it would be necessary to maintain a continuous minimum base flow through the DAF system. Therefore, even when return flows are not needed to maintain filter flows as desired, return flows would still occur and then be recycled back to the maturation ponds. This recycling of flows between the maturation ponds and DAF would be inefficient.

A key benefit of the sidestream concept is that the required capacity of the DAF system would be lower than that for the mainstream alternative. The DAF capacity would be the same as that of the Maturation Pond Effluent Pump Station discussed above for each concept.

Assuming the design peak equalized flow to the tertiary treatment system would be the same for both the mainstream and sidestream concepts, the maturation pond equalization volume needed would be the same. However, since most secondary effluent would bypass the maturation ponds in the sidestream configuration, priority pollutant dilution would not be provided to any significant extent. Eliminating this as an objective would mean that most of the maturation pond volume could be used for equalization storage. For the sidestream configuration, the desired minimum pool volume would be determined by operational considerations such as avoiding stagnation and minimizing algae growth. Similarly, for the mainstream configuration, if priority pollutant dilution is eliminated as an objective (at least during the wet season), most of the maturation pond volume would be available for equalization storage for this concept also, but would require higher pumping heads for the maturation pond return flow.

A conceptual comparison of the mainstream and sidestream maturation pond alternatives is presented in Table 2-1.

Table 2-1 Summary Comparison of Maturation Pond Mainstream and Sidestream Alternatives

Consideration	Mainstream	Sidestream
Priority Pollutant Dilution Provided?	Yes	No
Natural Disinfection Provided in the Maturation Ponds	Yes	Mostly no.
Effluent Cooling Provided	Yes	Mostly no.
Secondary Process Backup Provided	Yes	Mostly no.
Maturation Pond Feed Pump Station Capacity	50.0 Mgal/d (at 8 Mgal/d ADWF) (a)	Much smaller, depending on tertiary treatment capacity. However, may want to retain flexibility to pump all secondary effluent to the maturation ponds, in which case the required capacity would be the same as for the mainstream alternative.
Maturation Pond Effluent Pump Station Capacity	Same as tertiary treatment capacity.	Much smaller, depending on desired maximum drawdown rate for the maturation ponds.
Dissolved Air Flotation System Capacity	Same as tertiary treatment capacity.	Much smaller, depending on desired maximum drawdown rate for the maturation ponds
Maturation Pond Volume Available for Flow Equalization	Minimum requirement as determined by peak flow analysis. However, if the priority pollutant dilution objective is eliminated, then most of the pond volume would be available.	Most of the pond volume.
DAF, Filter, and UV Systems Equalized Flow Control	Simple – just control maturation pond outflow.	Complex – coordinated control of four flow rates, involving three pump systems and flow recycling between the maturation ponds and DAF.

(a) Maturation Pond Feed Pump Station capacity to be determined - see footnote (a) under Table 5-1 later in this report.

2.2 DETERMINATION OF MATURATION POND EQUALIZATION VOLUME REQUIREMENTS FOR THE MAINSTREAM ALTERNATIVE

The future amount of maturation pond volume required for equalization storage was determined by performing water balance calculations for the ponds under future flow conditions as described below. The methods used are generally the same as used for the 2012 PDR and the 2017 BODR. However, the calculations have been updated based on recent plant data.

A design maturation pond influent flow hydrograph was synthesized based on actual historical flows from June 1, 2016 (after connection of Placer County SMD1) through January 31, 2023. For each day in that period, the actual plant influent flow was converted to an equivalent future flow when the average dry weather flow is 8.0 Mgal/d. In the conversion, the increment by which an actual daily flow exceeded the average dry weather flow at that time (an indication of infiltration and inflow) was adjusted to an equivalent incremental flow for the future condition by assuming that the percent increase in this excess flow would be half of the percent increase in the average dry weather flow. Although the actual rate of increase of infiltration and inflow is uncertain and engineering judgement is required in future flow projections, the concept that infiltration and inflow should increase at a lower rate than the average dry weather flow makes logical sense because most of the backbone sewage collection system that would contribute to future infiltration and inflow is already existing and future sewers added should have relatively lower infiltration and inflow. For a hypothetical example of how future flows were calculated, consider the following: if on a given day in the historical database the influent flow to the plant was 6 Mgal/d when the average dry weather flow at that time was 4.0 Mgal/d, then the excess flow was 2 Mgal/d. For the future synthetic flow hydrograph, the average dry weather flow would increase by 100 percent to 8.0 Mgal/d and the excess flow would increase by 50 percent (half the average dry weather flow increase) from 2 Mgal/d to 3 Mgal/d, resulting in a total flow of 8+3=11 Mgal/d for the corresponding day in the future flow hydrograph. Any flows from the actual historical database that were less than or equal to the average dry weather flow at the time were converted to an equivalent future flow of 8.0 Mgal/d. It is realized that presuming all future flows would be at or above the design average dry weather flow over-estimates the flows during low flow periods. However, that is not important, because the evaluation of equalization requirements presented herein is based on high flow periods.

In addition to the synthesized plant influent flows described above, additional daily inputs to the maturation ponds included rainfall (when applicable) and plant recycle flows. Rainfall on the mechanical treatment plant site and on the maturation ponds were calculated based on the actual historical rainfall amounts recorded at the Lincoln plant site. Recycle flows to the maturation ponds were assumed to be 10 percent of the synthesized plant influent flow.

When the total influent flow to the maturation ponds exceeded the maturation pond effluent flow established for a particular scenario, the difference was stored in the maturation ponds. When the influent flow was less than the effluent flow, water was removed from maturation ponds.

The water balance calculations were based on daily flows, without consideration of diurnal variations. Theoretically, the storage volume required to equalize diurnal flow variations would be additive to the storage volume required to equalize daily average flows over a long-term peak flow event. However, the volume required to equalize diurnal variations is much lower than the volume required for long-term peak flow event equalization. The volume required for diurnal equalization will depend on the shape of the daily influent flow hydrograph, which will in turn vary with rainfall amounts throughout the day on peak flow days. Typically, the volume required to equalize the flow on a particular day can be expected to be around 15 percent of the total flow volume for that day. Based on the water balance calculations developed for this analysis, the maximum daily influent flow to the maturation ponds (including plant recycle flows and rainfall on the plant site and maturation ponds) for the years considered was 42 Mgal/d, which occurred in projected future conditions corresponding to an actual peak day plant influent flow of 20.8 Mgal/d and rainfall of 2.86 inches on December 31, 2022. Based on this extreme condition, the maximum diurnal equalization volume would be estimated at about 6 Mgal, which is at least an order of magnitude lower than volume requirements for long-term peak flow equalization developed in this study.

For this analysis, the maturation pond effluent flow was selected to match possible filter capacity, depending on the number of filter cells considered. Currently there are six filter cells, each with a capacity of 2.76 Mgal/d (based on a loading rate of 5 gpm/ft^2). Assuming one cell to be out of service, results in a current filter system reliable capacity of 13.80 Mgal/d. Filter system reliable capacities with possible additional cells are shown in Table 2-2.

Table 2-2 Filter System Reliable Capacity

Total Number of Filter Cells	Reliable Filter Capacity with One Cell Out of Service, Mgal/d
6	13.80
7	16.56
8	19.32
9	22.08
10	24.84

Figure 2-2 shows the storage volume required for flow equalization through the various peak flow events occurring in projected future conditions corresponding to the years evaluated. The maximum long-term peak flow equalization storage requirement for the mainstream alternative (before consideration of an appropriate safety factor and allowance for diurnal flow equalization) of 77 Mgal occurred as a result of storm conditions in December 2022 and January 2023, with the second highest event requiring only about 50 Mgal as a result of conditions occurring in March 2017.

Figure 2-3 shows the daily rainfall amounts and equalization storage volumes associated with the peak flow event in December 2022 and January 2023.

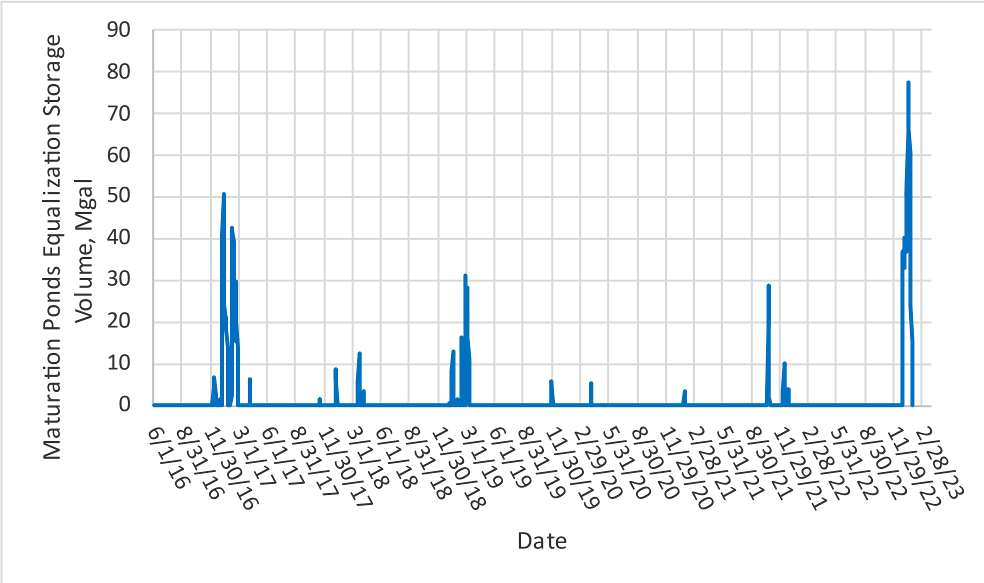

Figure 2-2 Future (8 Mgal/d ADWF) Maturation Pond Equalization Storage Volume Required Based on Maximum Maturation Pond Outflow of 19.32 Mgal/d for the Mainstream Alternative (Excludes Safety Factor and Diurnal Storage Allowance)

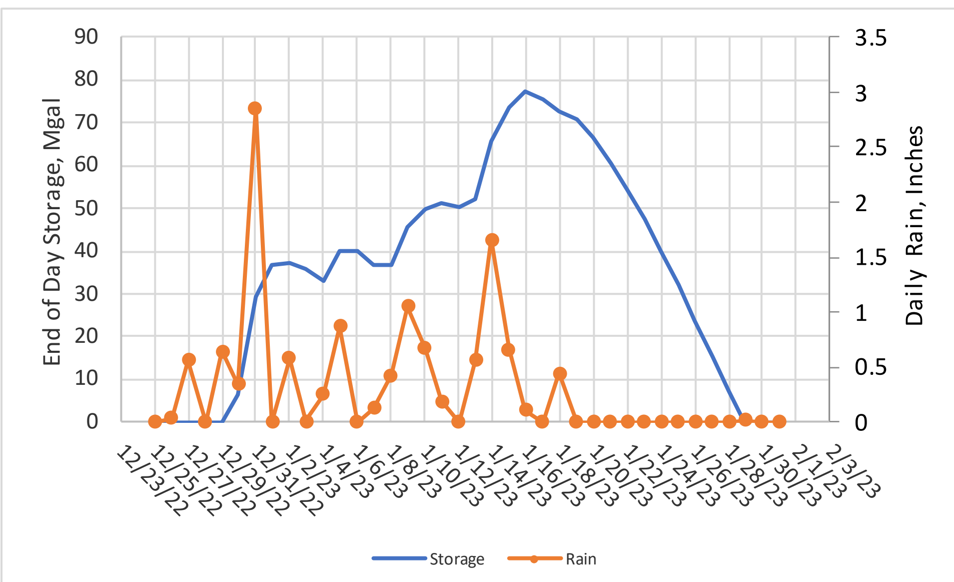

Figure 2-3 Future (8 Mgal/d ADWF) Daily Rainfall and Maturation Pond Storage Corresponding to Storm Event in December 2022 and January 2023 Based on Maximum Maturation Pond Outflow of 19.32 Mgal/d for the Mainstream Alternative (Excludes Safety Factor and Diurnal Storage Allowance)

During the 30 days prior to and including the day for which the future maximum equalization storage requirement of 77 Mgal occurred, the total rainfall was 11.64 inches, which is estimated to be around a 6-year return frequency. This return frequency, however, is based on Department of Water Resources data from 1947 to 2005 for a station in Lincoln (DWR Station A00-4947) that is no longer active. It is not known how plant data would correlate with data for the DWR station if it were still active. Therefore, the return frequency for the Lincoln plant site could be somewhat different.

A sensitivity analysis was completed to determine how the maximum equalization storage requirement would vary based on the maximum maturation pond effluent flow (filtration system flow). The results are shown in Table 2-3. It would be appropriate to apply a safety factor to the indicated maturation pond equalization volumes to account for uncertainties in the analysis and for possible more severe storm events that occurred in the period studied. Also, a diurnal storage allowance should be added. The last column in Table 2-3 shows suggested design volumes with these additional considerations. Since the total existing maturation pond volume is 177 Mgal and the volume available for equalization storage would be much lower, it seems clear that at least 8 filter cells (reliable capacity = 19.32 Mgal/d) should be considered for the future expansion to 8 Mgal/d average dry weather flow.

Based on 8 filter cells, the existing maturation ponds, and the suggested equalization storage volume shown in Table 2-3, the minimum volume available for priority pollutant dilution would be 177-103=74 Mgal (water surface elevation = 103.8 ft). This volume would provide hydraulic retention times of 3.8 days at the peak tertiary flow of 19.32 Mgal/d and 8.4 days at 8.8 Mgal/d (the future ADWF plus 10% recycle allowance). Since priority pollutant dilution is not likely to be a significant issue during peak flows, the lower hydraulic retention time in that case is not concerning.

To provide maximum operational flexibility, if determined to be reasonably possible during detail design, the ability to pump the maturation ponds down to a depth of about 5 feet (water surface elevation of 101.3 ft) should be provided, resulting in an available equalization storage volume of 129 Mgal.

Table 2-3 Mainstream Maturation Pond Equalization Volume Sensitivity to Maximum Maturation Pond Effluent Flow (Based on 8 Mgal/d ADWF)

Total Number Filter Cells	Reliable Filter Capacity and Maximum maturation pond Effluent Flow, Mgal/d	Maturation Pond Equalization Volume without Safety Factor or Diurnal Storage, Mgal	Suggested Maturation Pond Equalization Volume with Safety Factor and Diurnal Storage Allowance (a), Mgal
6	13.80	229	292
7	16.56	129	167
8	19.32	77	103
9	22.08	40	55
10	24.84	29	42

(a) Based on safety factor of 1.25 and diurnal equalization storage volume = 6 Mgal.

2.3 DETERMINATION OF MATURATION POND EQUALIZATION VOLUME REQUIREMENTS FOR THE SIDESTREAM ALTERNATIVE

The water balance calculations for the sidestream alternative followed the same procedures as those for the mainstream alternative, with the following exceptions:

1. Secondary effluent flows (including recycle flows and rainfall on the mechanical treatment plant site) were routed directly to the filtration system, up to the capacity of that system established for each scenario.

2. Secondary effluent flows in excess of tertiary treatment capacity were routed to the maturation ponds.

3. When secondary effluent flows were reduced lower than the tertiary treatment capacity, return flows from the maturation ponds were provided to maintain the total flow through the tertiary treatment system at capacity, subject to return flow capacity limitations considered in various scenarios.

When the maturation pond return flow capacity was not limited below the amount required to sustain the tertiary treatment flow at its capacity, equalization storage requirements were exactly the same as required for the mainstream alternative (see Figures 2-2 and 2-3 and Table 2-3). The daily average return flows that occurred when the tertiary treatment capacity was set to 19.32 Mgal/d are shown in Figure 2-4. However, in many cases these return flows could have been reduced without substantially impacting equalization storage volume requirements, as discussed below.

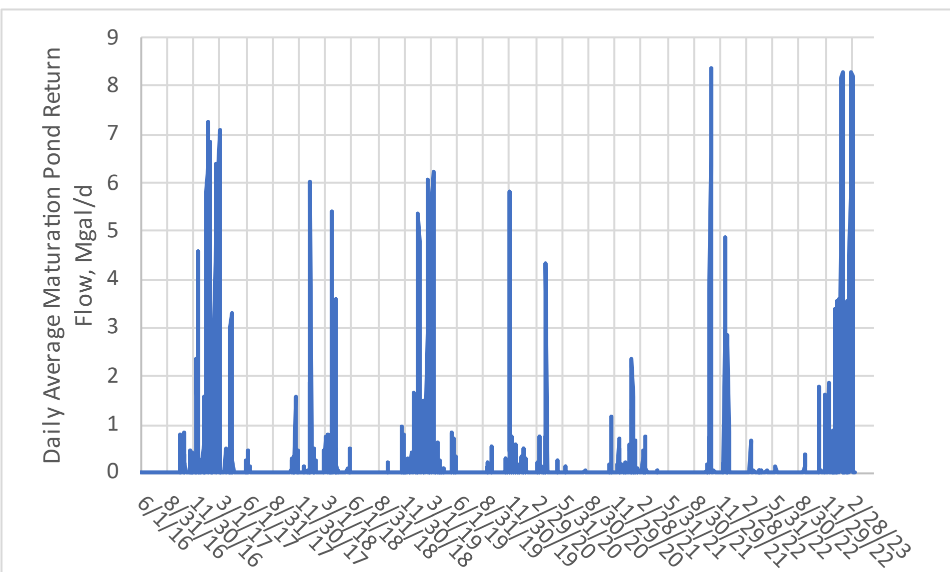

Figure 2-4 Daily Maturation Pond Average Return Flows for Tertiary Treatment Capacity of 19.32 Mgal/d for the Sidestream Alternative (Based on 8 Mgal/d ADWF)

A sensitivity analysis was completed to determine how maturation pond return flow capacity could impact the maturation pond equalization storage requirements. The impact of reducing the return flow is to slow the drainage of the maturation ponds after a peak flow event. For widely spaced storms, such as mostly occurred in the study period, impacts would be minimized because a longer time for drainage would still be completed before the next storm event occurred. This is illustrated in Figure 2-5 that shows almost no impact on the required storage volume when the maturation pond return flow capacity is limited to 3 Mgal/d (daily average basis) for the December 2022 / January 2023 event.

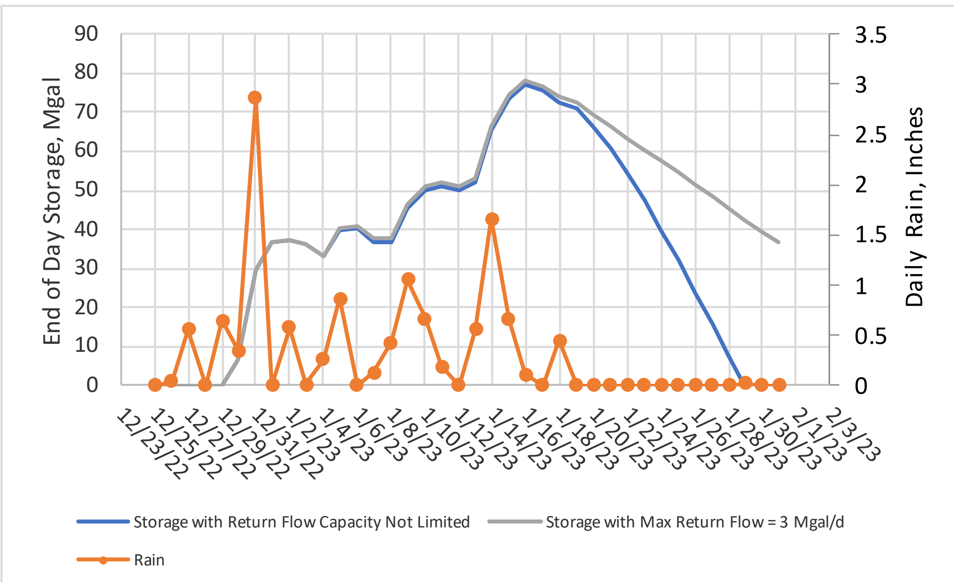

Figure 2-5 Future (8 Mgal/d ADWF) Daily Rainfall and Maturation Pond Storage Corresponding to Storm Event in December 2022 and January 2023 Based on Variable Maximum Maturation Pond Return Flows for the Sidestream Alternative

If potential back-to-back events occurred, the maturation ponds might not be fully drained before beginning to fill again if maturation pond return flows are restricted. This is illustrated in Figure 2-6, which is based on a hypothetical event in which plant flows and rainfalls for the December 2022 and January 2023 event were repeated almost immediately after the maturation pond would be fully drained with return pumping capacity adequate to sustain tertiary treatment at full capacity. In the figure, equalization volumes that would occur if the maturation pond return flow rate was limited to 3 Mgal/d are contrasted with equalization volumes without that 3 Mgal/d limit. As indicated in the figure, the maximum equalization storage capacity was drastically increased to from 77 Mgal to 112 Mgal in the hypothetical case when the return flow was limited.

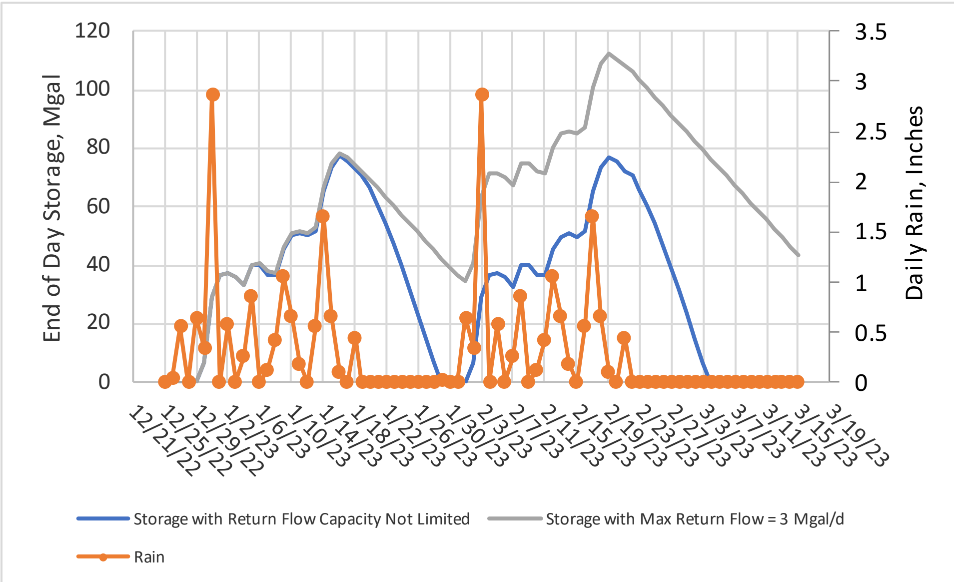

Figure 2-6 Future (8 Mgal/d ADWF) Daily Rainfall and Maturation Pond Storage Corresponding to Hypothetical Back-to-Back Storms Like the Event in December 2022 and January 2023 Based on Variable Maximum Maturation Pond Return Flows for the Sidestream Alternative

The impact of maximum return flows on maximum equalization storage volume were further evaluated in a sensitivity analysis for three actual events and the hypothetical event described above. The results are shown in Figure 2-7. As shown in the figure, maximum equalization storage volumes were not significantly impacted by maximum maturation pond return flow capacities greater than 2.5 Mgal/d for the actual events. However, for the hypothetical back-to-back storms, storage requirements were increased when the maturation pond return flow capacity was limited to less than 5.5 Mgal/d.

It must be recognized that maturation pond return flows considered in the evaluations discussed above are daily averages and that diurnal variations in flow were not considered. During maturation pond drawdown, plant influent flows and secondary process flows would typically remain elevated above dry weather flows due to the lingering effects of the preceding storm event (continued infiltration and inflow). A reasonable allowance is to assume that the daily average secondary effluent flow during maturation pond drawdown could be 150 percent of the future average dry weather flow (12 Mgal/d for the future 8 Mgal/d ADWF condition). Since diurnal minimum flows could be perhaps half of the daily average, the maturation pond return flows needed to sustain tertiary treatment flows at capacity throughout each day would be about 6 Mgal/d (50% of 12 Mgal/d) higher than considered above without diurnal variation. Therefore, the return capacity needed to avoid increasing equalization storage capacity would be 2.5+6=8.5 Mgal/d and 5.5+6=11.5 Mgal/d for the actual storm events and the hypothetical storm event, respectively, considered in Figure 2-7. However, depending on the actual shapes of daily secondary process flow hydrographs during maturation pond drawdown, it is likely that flows somewhat lower than the 8.5 Mgal/d and 11.5 Mgal/d could be used without significantly impacting maximum maturation pond equalization storage requirements. A reasonable design value of 10 Mgal/d is suggested for the 8 Mgal/d average dry weather flow scenario. Depending on detail design considerations based on actual pump selections, some flexibility in the design flow may be appropriate.

A hydraulic analysis of the existing submersible pump system used for draining the maturation ponds indicates the ability to pump up to 9.5 Mgal/d with a pond water surface elevation of 103.8, if about 40 feet of combined 12-inch discharge piping is replaced with parallel piping. This decreases to about 9.1 Mgal/d if the maturation pond water surface elevation is lowered to 101.3 ft. Although lower than the 10 Mgal/d recommendation, these pumping rates may be reasonably acceptable. To reach the 10 Mgal/d target, it is likely that minor modifications would be required (perhaps changing impellers or over-speeding the pumps).

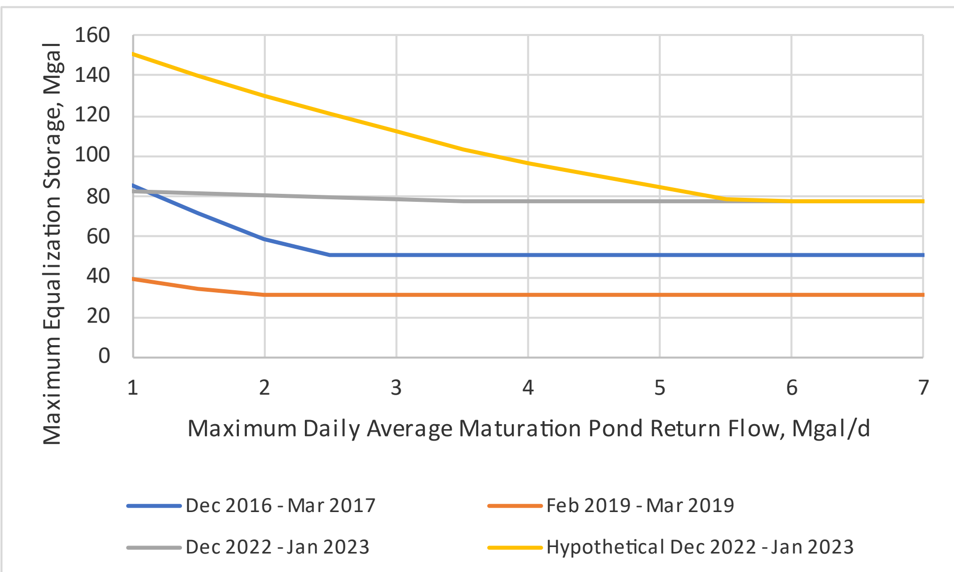

Figure 2-7 Effect of Maximum Maturation Pond Return Flow on Maximum Maturation Pond Equalization Volume for the Sidestream Alternative Based on Tertiary Treatment Capacity of 19.32 Mgal/d (Based on 8 Mgal/d ADWF)

3.0 UPDATED EVALUATION OF TERTIARY STORAGE REQUIREMENTS

Beginning in March 2018, continuous on-line monitoring of Auburn Ravine Creek flows and temperatures (both upstream and downstream of the WWTRF discharge) was started. This real-time data is now used in the plant supervisory control and data acquisition (SCADA) system to automatically control the plant discharge. Furthermore, the historical flow and temperature recordings were used in this updated evaluation of tertiary storage requirements. Although continuous on-line data were available for four complete water years (each including October through the following September), the temperature recordings for Auburn Ravine Creek upstream of the plant effluent were compromised in Water Year 2021 (ending September 30, 2021). Therefore, this analysis includes evaluations for Water Years 2019, 2020, and 2022. For each of those years, calculations were made to evaluate hypothetical conditions if the same creek flows and temperatures and discharge temperatures that occurred in that year occurred again in future years when plant flows reach 8 Mgal/d ADWF.

For each water year considered, two sets of analyses were completed; one in which the discharge temperature was presumed to be the actual effluent temperature recorded for the year in question and one in which the discharge temperature was presumed to be the temperature recorded in the oxidation ditches for that year. Using recorded effluent temperatures represents conditions when the plant secondary effluent is routed through the maturation ponds prior to tertiary treatment and discharge, as is the typical current practice (maturation pond mainstream alternative). Using oxidation ditch temperatures for the discharge allowed evaluation of potential future operations in which most of the secondary effluent would be routed directly to tertiary treatment and discharge, without going through the maturation ponds (maturation pond sidestream alternative). However, even if most of the secondary effluent were to be routed directly to tertiary treatment and discharge, diurnal peak flows and excess peak wet weather flows would still be routed through the maturation ponds for equalization. Since both the maturation ponds and tertiary storage basins result in cooling of the wastewater (except perhaps in some warm months when the effluent is used for agricultural irrigation), assuming that the discharge would be at the temperature of the oxidation ditches, despite return flows from the maturation ponds and tertiary storage basins (when applicable), is a conservative boundary condition – actual temperatures would be lower.

Every 15 minutes for each water year various calculations were made based ambient temperatures and flows in Auburn Ravine Creek and wastewater discharge temperatures. In each 15-minute time increment, the maximum allowable discharge, estimated actual discharge, estimated diversion to (or return flow from) the tertiary storage basins, and potential volume stored in the tertiary storage basins were calculated based on the most limiting of nine criteria:

1. The discharge shall not cause the creek temperature to rise more than 15 °F above background creek temperature (see discussion below).

2. During October through December, the discharge shall not cause the creek temperature downstream from the WWTRF discharge to rise above 67 °F.

3. During January through May, the discharge shall not cause the creek temperature downstream from the WWTRF discharge to rise above 63 °F.

4. During October through May, if the background creek temperature was already above the limit of 64 °F or 68 °F, as applicable, the temperature rise caused by the discharge is limited to 4 °F.

5. The discharge shall not exceed 25 Mgal/d.

6. Except when storage return flows are applicable, the discharge shall not exceed the projected future monthly average influent (including infiltration and inflow) flow plus the monthly average rainfall accumulation for all plant facilities (rain catchment area used was 145 acres).

7. The discharge shall be zero during the months of June through August.

8. As applicable, when storage return flows were possible, the discharge was limited by the residual potential storage volume in the tertiary storage basins at the time of complete drawdown.

9. To prevent switching between diversions to storage and return from storage multiple times daily, no return was allowed unless there were no diversions in the previous five days.

Except as noted below, each of the triggering temperatures listed above is 1 °F lower than the corresponding permit limits. This is intended to provide a safety margin to assure permit compliance.

As noted in Item 1 above, in this analysis the discharge was allowed to cause the creek temperature to increase up to 15 °F above background creek temperature; however, this condition was applicable only when other criteria were not more stringent (e.g., Items 2, 3, and 4). The permit allows an annual average increase of up to 5 °F. Allowing an increase of up to 15 °F on certain days (when other criteria are less stringent or not applicable) may be possible because the days of high temperature increase would be offset by many days of lower temperature increase or no temperature increase in an annual average. Particularly, it is noted that there are several months (at least June through August and potentially May and September) when all effluent could be routed to agricultural irrigation instead of discharge to the creek. However, to gain credit for a day of no temperature impact on the creek, a minor amount of discharge may be necessary; perhaps 1,000 gallons, which would not measurably impact creek temperatures. This analysis includes calculation of the average annual temperature increase to confirm that the 5 °F criterion can be met.

It was necessary to determine when the actual discharge would be less than the maximum allowable discharge, since using the maximum allowable discharge would inappropriately skew the temperature impact on the creek. This is the reason for Item 6 above.

Although the analysis forced zero discharge to the creek in June through August (Item 7), a small discharge that would not measurably impact creek temperature may be required as noted above.

Diversions to the tertiary storage basins were calculated when the allowable discharge was less than the projected future average monthly influent flow (including infiltration and inflow) plus rain captured on/in plant facilities. The projected monthly average influent flows and rain capture were determined specifically for each water year based on actual plant influent flows in that water year transformed to future 8 Mgal/d ADWF conditions (see discussion under maturation pond analysis) and based on actual rainfall amounts in those water years. Estimated return flows from the tertiary storage basins, when applicable, were calculated as the maximum allowable discharge minus the monthly average influent flow and rain capture. These return flows are indicated as negative diversion flows in the calculations and results presented below. Cumulative inflows and outflows for the tertiary storage basins were used to determine the potential volume in the tertiary storage basins during each time step.

The analysis of discharges and tertiary storage basin conditions did not consider the possibility of agricultural irrigation using water from the tertiary storage basins. Instead, it was assumed that all water accumulated would be available for return flow and discharge to the creek, except during the months of June through August, when there was no discharge to the creek. In June through August, all water in the tertiary storage basins would be used for agricultural irrigation. Except for June through August, the assumption that all stored water would be returned for creek discharge when possible resulted in conservatively high estimates of discharge flows whenever return flows were indicated. This, in turn, resulted in conservatively high estimates of creek temperature impacts. Because the possibility of using tertiary storage basin contents for agricultural irrigation was not considered, the tertiary storage basin volumes calculated in this analysis were potential maximum volumes. Estimated actual volumes in the tertiary storage basins were determined in subsequent water balance calculations, which are discussed later in this document.

Because all current effluent flows and any tertiary storage basin volume remaining on or after June 1 each year would be used for agricultural irrigation, the potential tertiary storage volume was forced to zero on June 1 in each scenario analyzed to prevent basin drawdown by discharge to the creek in the calculations. In reality, the basin would be drawn down gradually, not suddenly, as the water is used for agricultural irrigation.

3.1 WATER YEAR 2019 ANALYSIS

Calculated flows and potential storage volumes for Water Year 2019 are shown in Figure 3-1 and Figure 3-2, representing discharge at effluent temperatures and discharge at oxidation ditch temperatures, respectively. As shown for both cases, diversions to the tertiary storage basins were required in the Fall and Spring, but not in the winter. As would be expected, more diversions were required and more potential storage occurred with the discharge at oxidation ditch temperatures than with the discharge at effluent temperatures, although the differences were much more pronounced in the Fall than in the Spring. The maximum potential storage was 32 Mgal and 194 Mgal, respectively.

Calculated creek temperatures for Water Year 2019 are shown in Figure 3-3 and Figure 3-4, representing discharge at effluent temperatures and discharge at oxidation ditch temperatures, respectively. As would be expected, oxidation ditch temperatures resulted in substantially higher temperatures in the creek downstream from the discharge (Station R2) and higher temperature changes in the creek (R2-R1). Annual average temperature changes were 1.86 °F and 3.81 °F, respectively, indicating the acceptability of allowing temperature changes up to 15 °F in the creek during times when other limitations are less restrictive. The 15 °F threshold could be adjusted as desired and appropriate for actual operations.

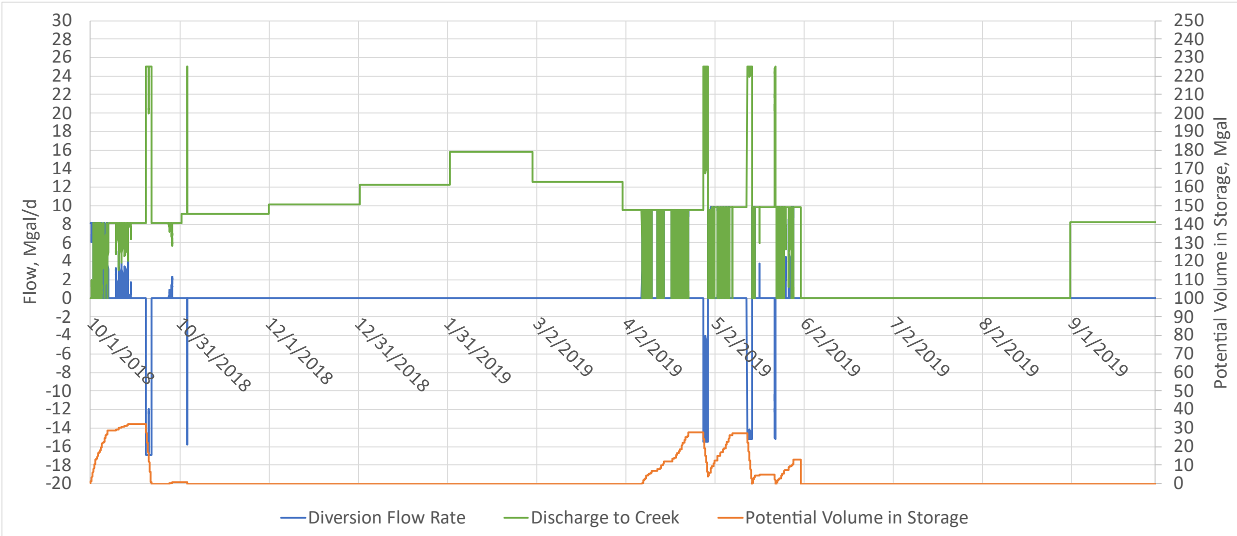

Figure 3-1 Water Year 2019 Flows and Storage with Discharge at Effluent Temperatures (Flows Transformed to Future 8 Mgal/d ADWF Condition)

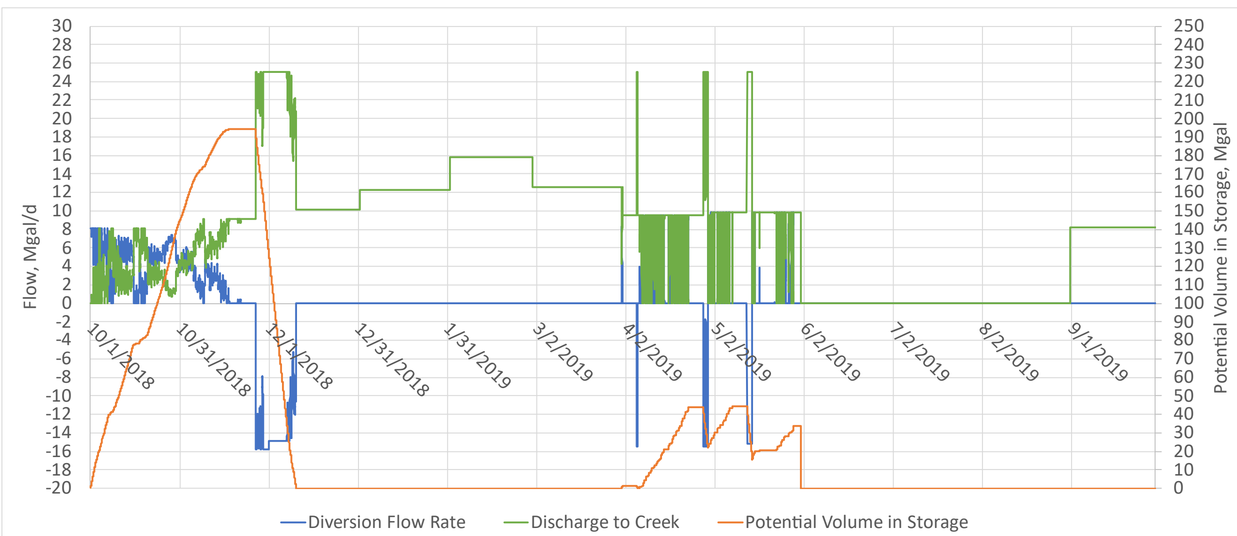

Figure 3-2 Water Year 2019 Flows and Storage with Discharge at Oxidation Ditch Temperatures (Flows Transformed to Future 8 Mgal/d ADWF Condition)

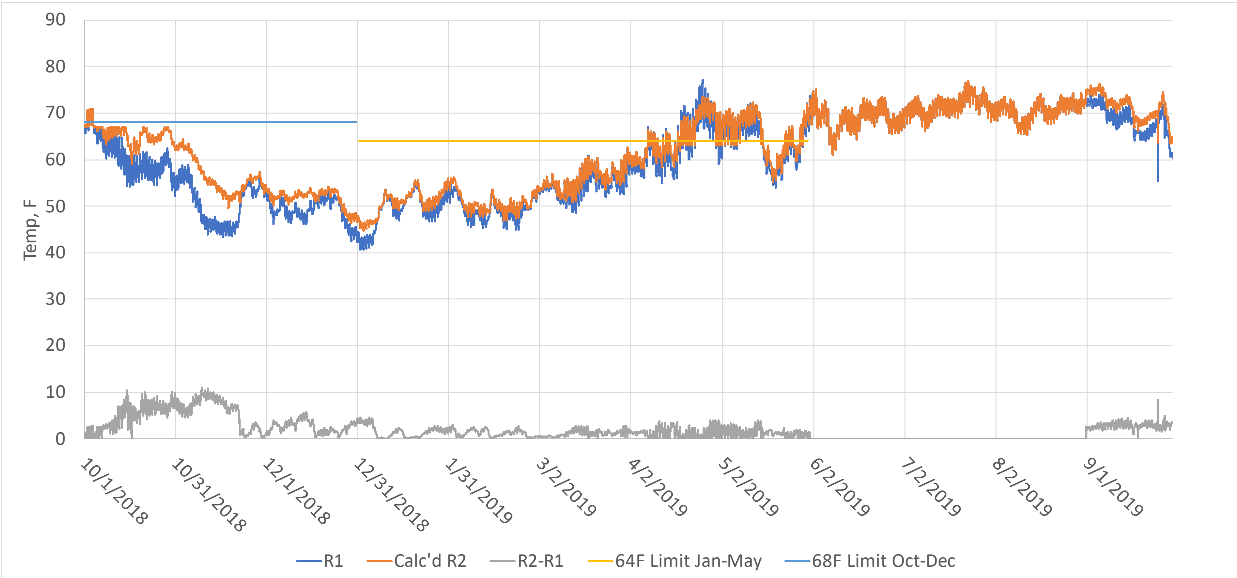

Figure 3-3 Water Year 2019 Creek Temperatures with Discharge at Effluent Temperatures (Effluent Flows Transformed to Future 8 Mgal/d ADWF Condition)

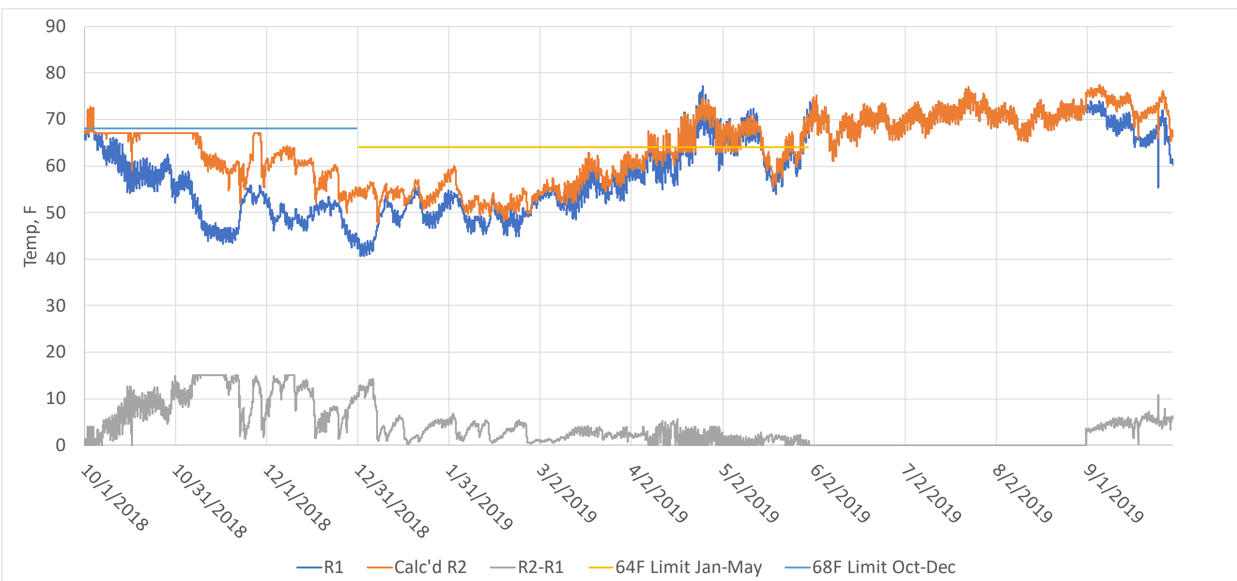

Figure 3-4 Water Year 2019 Creek Temperatures with Discharge at Effluent Temperatures (Effluent Flows Transformed to Future 8 Mgal/d ADWF Condition)

3.2 WATER YEAR 2020 ANALYSIS

Calculated flows and potential storage volumes for Water Year 2020 are shown in Figure 3-5 and Figure 3-6, representing discharge at effluent temperatures and discharge at oxidation ditch temperatures, respectively. As shown in Figure 3-6, many diversions were required and much potential storage was accumulated in October and November for the scenario with oxidation ditch temperatures, while no diversions and storage were indicated in the Fall with effluent temperatures. Diversions and storage in the Spring were relatively minor for both effluent and oxidation ditch temperatures. The maximum potential storage was 39 Mgal (in the Spring) and 159 Mgal (in the Fall), respectively.

Calculated creek temperatures for Water Year 2020 are shown in Figure 3-7 and Figure 3-8, representing discharge at effluent temperatures and discharge at oxidation ditch temperatures, respectively. Again, as would be expected, oxidation ditch temperatures resulted in substantially higher temperatures in the creek downstream from the discharge (Station R2) and higher temperature changes in the creek (R2-R1). Annual average temperature changes were 2.34 °F and 4.90 °F, respectively. The 4.90 °F annual average temperature change indicated when oxidation ditch temperatures were used seems perhaps too close to the 5 °F permit limit. However, as explained previously, these temperature changes are overestimated because they don't recognize the benefits of a portion of the flow being cooled in the maturation ponds and tertiary storage basins.

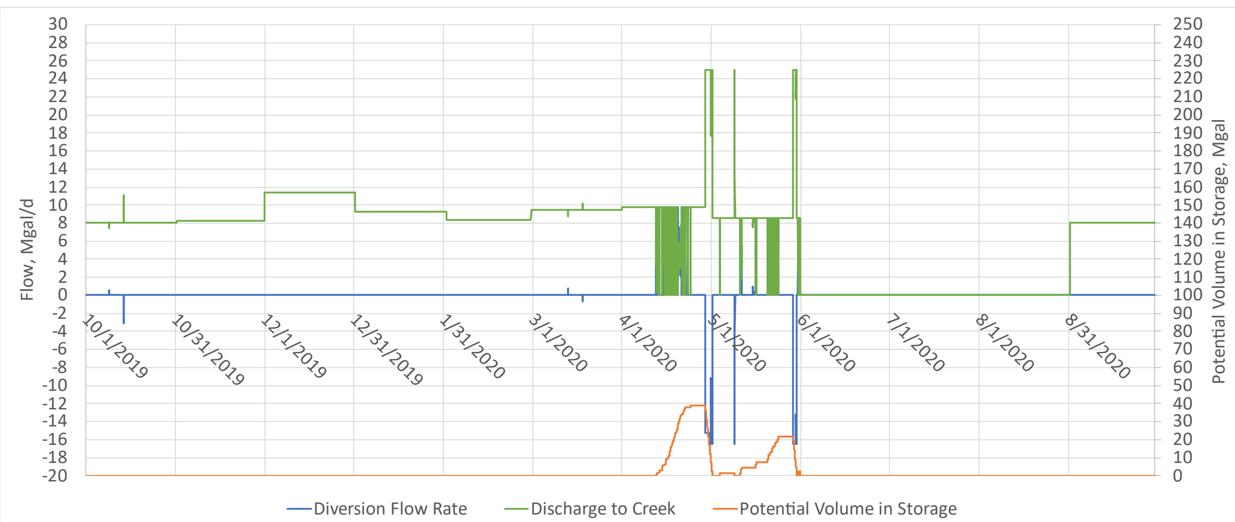

Figure 3-5 Water Year 2020 Flows and Storage with Discharge at Effluent Temperatures (Flows Transformed to Future 8 Mgal/d ADWF Condition)

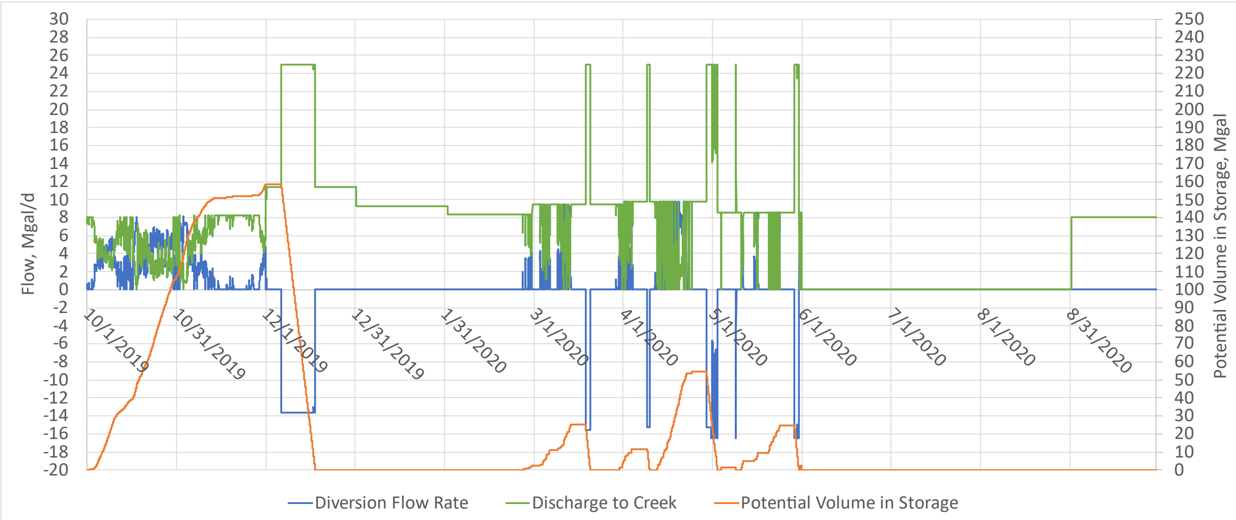

Figure 3-6 Water Year 2020 Flows and Storage with Discharge at Oxidation Ditch Temperatures (Flows Transformed to Future 8 Mgal/d ADWF Condition)

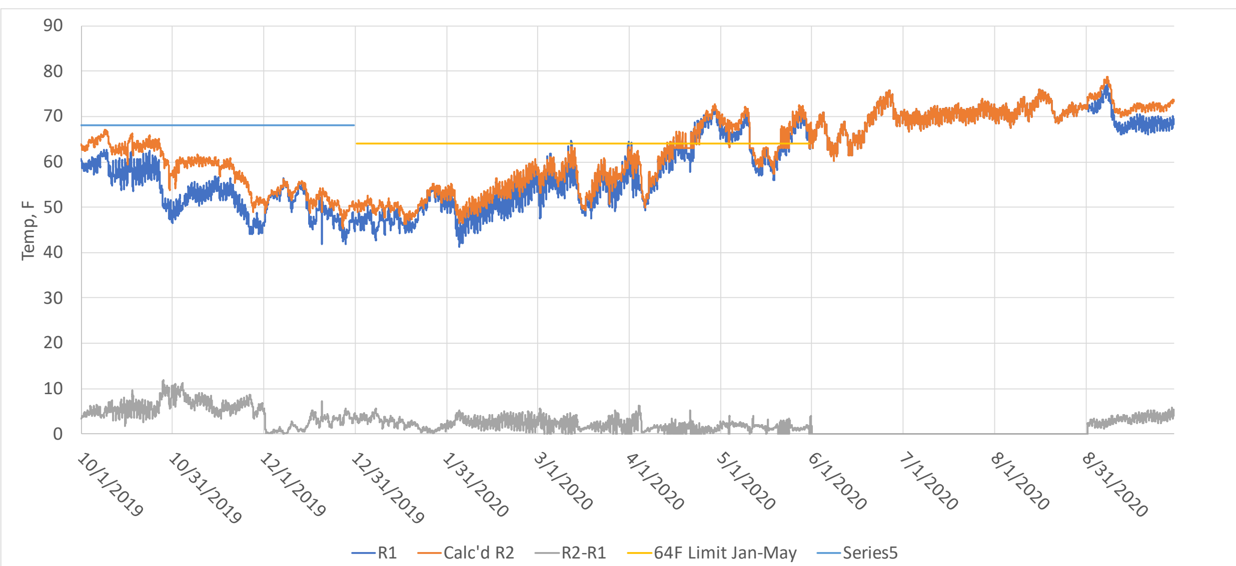

Figure 3-7 Water Year 2020 Creek Temperatures with Discharge at Effluent Temperatures (Effluent Flows Transformed to Future 8 Mgal/d ADWF Condition)

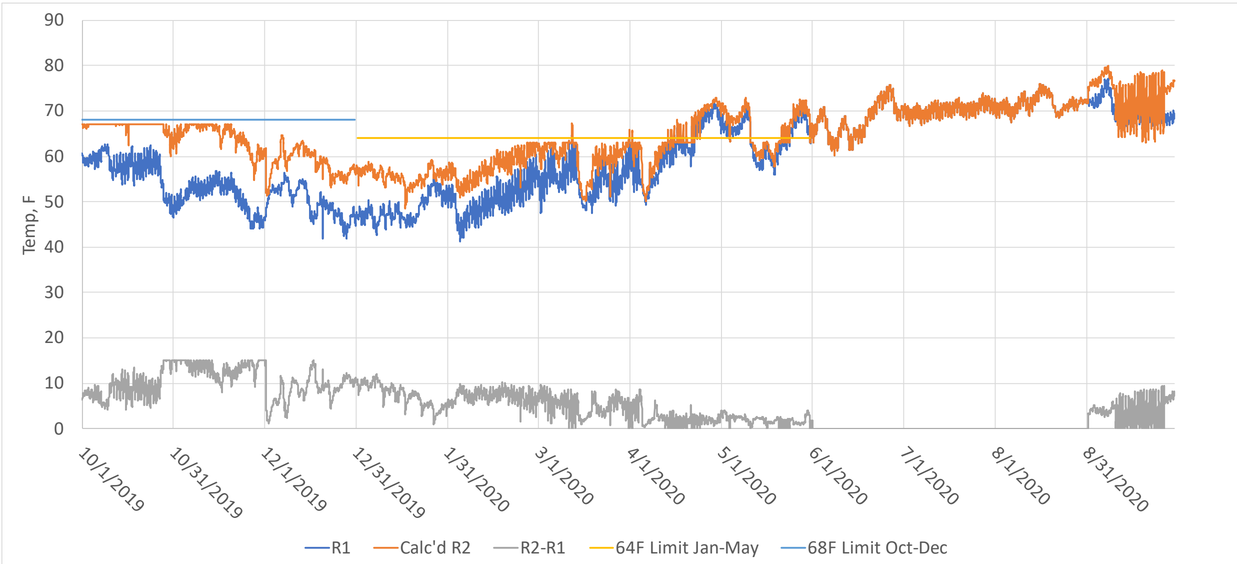

Figure 3-8 Water Year 2020 Creek Temperatures with Discharge at Oxidation Ditch Temperatures (Effluent Flows Transformed to Future 8 Mgal/d ADWF Condition)

3.3 WATER YEAR 2022 ANALYSIS

Calculated flows and potential storage volumes for Water Year 2022 are shown in Figure 3-9 and Figure 3-10, representing discharge at effluent temperatures and discharge at oxidation ditch temperatures, respectively. As shown in the figures, diversions to the tertiary storage basins occurred in both Fall and Spring. The maximum potential storage for both scenarios occurred in the Spring and were 164 Mgal and 249 Mgal for effluent temperatures and oxidation ditch temperatures, respectively.

Calculated creek temperatures for Water Year 2022 are shown in Figure 3-11 and Figure 3-12, representing discharge at effluent temperatures and discharge at oxidation ditch temperatures, respectively. Again, as would be expected, oxidation ditch temperatures resulted in substantially higher temperatures in the creek downstream from the discharge (Station R2) and higher temperature changes in the creek (R2-R1). Annual average temperature changes were 2.56 °F and 4.21 °F, respectively.

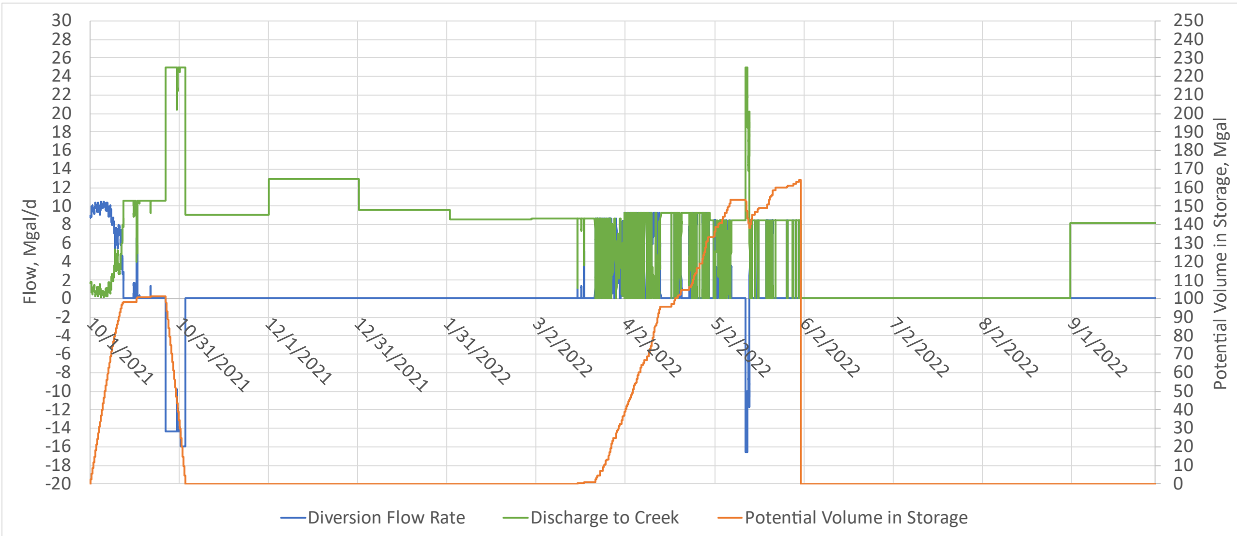

Figure 3-9 Water Year 2022 Flows and Storage with Discharge at Effluent Temperatures (Flows Transformed to Future 8 Mgal/d ADWF Condition)

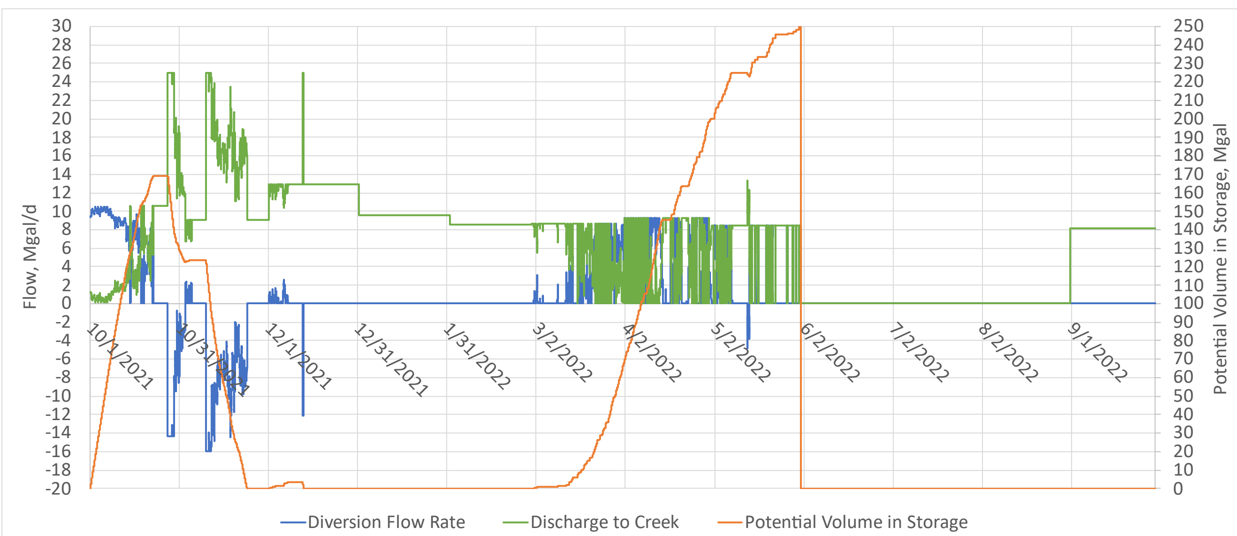

Figure 3-10 Water Year 2022 Flows and Storage with Discharge at Oxidation Ditch Temperatures (Flows Transformed to Future 8 Mgal/d ADWF Condition)

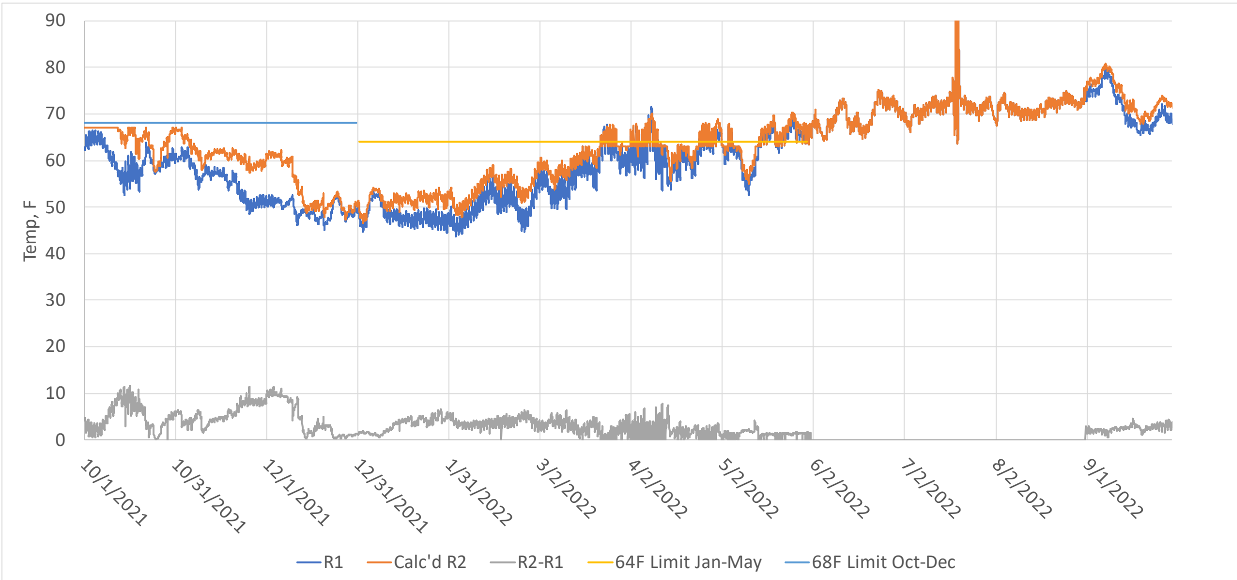

Figure 3-11 Water Year 2022 Creek Temperatures with Discharge at Effluent Temperatures (Effluent Flows Transformed to Future 8 Mgal/d ADWF Condition)

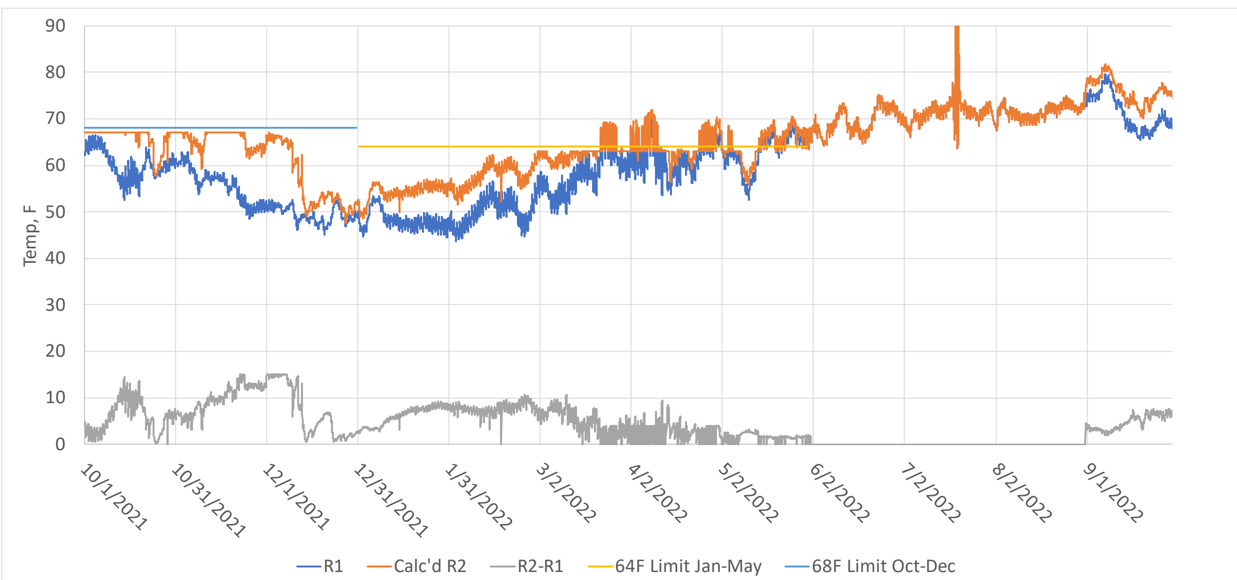

Figure 3-12 Water Year 2022 Creek Temperatures with Discharge at Oxidation Ditch Temperatures (Effluent Flows Transformed to Future 8 Mgal/d ADWF Condition)

3.4 SUMMARY OF ANALYSES FOR WATER YEARS 2019, 2020, AND 2022 WITHOUT CONSIDERATION OF WATER BALANCE CALCULATIONS

Results of the analyses for Water Years 2019, 2020, and 2022 presented above are summarized in Table 3-1.

Table 3-1 Summary of Results for Water Years 2019, 2020, and 2022 Without Consideration of Water Balance Calculations (Based on Plant Flows Transformed to Future 8 Mgal/d ADWF Condition)

Water Year	Discharge at Effluent Temperatures		Discharge at Oxidation Ditch Temperatures	
	Maximum Potential Storage, Mgal (a)	Annual Average Temperature Increase in Creek, °F	Maximum Potential Storage, Mgal (a)	Annual Average Temperature Increase in Creek, °F (b)
2019	32	1.86	194	3.81
2020	39	2.34	159	4.90
2022	164	2.56	249	4.21

(a) Actual storage requirements will be lower due to irrigation reuse as determined by water balance calculations discussed in the next section.
(b) Actual average annual temperature change in creek will be lower do to cooling of a portion of the plant flow in the maturation ponds and tertiary storage basins.

The calculations discussed and summarized above were based on a maximum allowable discharge of 25 Mgal/d, which is a permit requirement. Currently, the Effluent Pump Station has a reliable capacity of 20.4 Mgal/d and would have to be upgraded to match the permit limit. However, when a 20.4 Mgal/d discharge limit was included in the calculations (results not specifically presented), storage requirements were slightly increased at certain times of the year, but the maximum storage requirements were not impacted. Similarly, annual average temperature increases in the creek were not significantly impacted. Therefore, it is not necessary to increase the capacity of the Effluent Pump Station based on temperature limits or tertiary storage capacity. However, to maximize operational flexibility, it may be desirable to increase the capacity of the Effluent Pump Station to the permitted limit of 25 Mgal/d.

3.5 WATER BALANCE CALCULATIONS

The general methodology used for water balance calculations in this study is the same as described in the 2017 BODR, with the following important differences:

1. The input data for average monthly precipitation and reference evaporation (ET_0) are actual recorded values for the water year in question. Precipitation data was from plant records, while the reference evapotranspiration data is the average of values recorded for Davis, Fair Oaks, and Auburn obtained from the California Irrigation Management Information System (CIMIS).

2. The monthly average infiltration and inflow amounts are from daily transformations of actual plant flows occurring in the indicated water years to projected future conditions when the plant flow increases to 8 Mgal/d ADWF (see maturation pond analysis for further details).

3. The monthly average maximum discharge flows were the estimated monthly average discharge flows determined from the calculations discussed in Sections 3.1 through 3.4.

4. The rain catchment area for the mechanical plant site was included with the maturation pond rain catchment area.

5. The existing tertiary storage basin area and volume were held constant at current values and future storage requirements and required storage volumes were compared to the existing storage volume to indicate surplus storage volume available.

For all scenarios considered, the available area for agricultural reuse was held at 942 acres, which is the current value.

Since water balance calculations based on discharges at oxidation ditch temperatures would represent the most severe conditions, they are considered first. The corresponding water balances for Water Years 2019, 2020, and 2022 are shown in Appendix A. The tertiary storage requirements indicated in the water balances for Water Years 2019, 2020, and 2022 are 7, 6, and 92 Mgal, respectively. These relatively low requirements, when compared to the potential storage values shown in Table 3-1, resulted from irrigation reuse of water that was discharged to the tertiary storage basins in the calculations used to develop Table 3-1, preventing accumulation of any substantial storage volume. The 7 and 6 Mgal requirements determined for Water Years 2019 and 2020 were nuisance accumulations of rain in the tertiary storage basins. The tertiary storage basin volume of 92 Mgal indicated for Water Year 2022 occurred in the month of October.

The storage requirement of 92 Mgal occurring in October of Water Year 2022 when oxidation ditch temperatures were used was reduced to 1 Mgal in a corresponding water balance using effluent temperatures (water balance not presented in Appendix A). Similarly, by inspection, water balances for Water years 2019 and 2020 based on effluent temperatures would indicate no required storage (nuisance accumulations of rain in the tertiary storage basins disregarded).

The volume of tertiary storage needed for temperature compliance was determined in the 2017 BODR to be about 290 Mgal. Despite updated higher peak flows now being considered, the tertiary storage requirement for temperature compliance has been drastically reduced as a result of new permit temperature requirements. If the current practice of discharging effluent that has been cooled in the maturation ponds is continued (the mainstream alternative), essentially no tertiary storage would be needed for temperature compliance based on the three years of data analyzed for this study (however, a modest amount of storage [perhaps 50 Mgal] would be required for irrigation reuse operations). Even with sidestream maturation ponds, the maximum storage requirement for temperature compliance determined in this analysis is 92 Mgal, based on an agricultural irrigation area of 942 ac. Even if that area was reduced to 762 ac due to loss of the existing center pivot irrigation system, the storage requirement would increase to only 97 Mgal.

It must be emphasized that only three years of data have been evaluated based on newly available continuous recordings of creek flows and temperatures. Therefore, considerable conservatism is warranted. Since the existing tertiary storage basin volume is 190 Mgal, it is now apparent that no additional tertiary storage is required for plant expansion to 8.0 Mgal/d.

4.0 CONSIDERATION OF SMALLER INCREMENTAL EXPANSION

Phase 1 and Phase 2 design capacities of 7.1 and 8.0 Mgal/d average dry weather flow, respectively, established in the 2017 BODR were based on logical increments of expansion for the secondary treatment system – the addition of an oxidation ditch for Phase 1 and a clarifier for Phase 2. Given that the current average dry weather flow is only about 4.4 Mgal/d, such expansions would likely provide adequate plant capacity for many years, as illustrated in Figure 4-1. In the figure, four growth scenarios are considered: linear growth at the actual rate experienced from 2016 through 2022 and growth at annual rates of 1, 2, and 3 percent. Even at the relatively fast growth rate of 3 percent annually, the Phase 1 capacity of 7.1 Mgal/d would not be reached until about 2039, or about 16 years in the future.

In this section, the possible expansion of the maturation ponds and downstream facilities for something less than the Phase 1 and Phase 2 capacities mentioned above is considered. The objective is to determine if shorter-term and less costly improvements in facilities and/or operations to the maturation ponds and downstream facilities would make sense, while keeping in mind that expansion to 8.0 Mgal/d to match the upgraded secondary process capacity will eventually be required. Clearly, if substantial new physical facilities are required even for the lower capacity, it would not make sense to construct those features for the lower capacity unless they are also consistent with requirements at the future larger capacity.

The following criteria are suggested for evaluation of the appropriate design capacity for the next expansion of the maturation ponds and downstream facilities:

- Construction could be completed 2 years from the time of this report.
- Construction of a subsequent expansion could take 2 years.
- At least 5 years should be allowed between completion of construction for the next expansion and beginning of construction for the subsequent expansion.

On the basis of the criteria above, the next expansion of the maturation ponds and downstream facilities would be designed for the capacity required in mid-2032, which varies from about 4.8 to 5.8 Mgal/d for the growth scenarios shown in Figure 4-1.

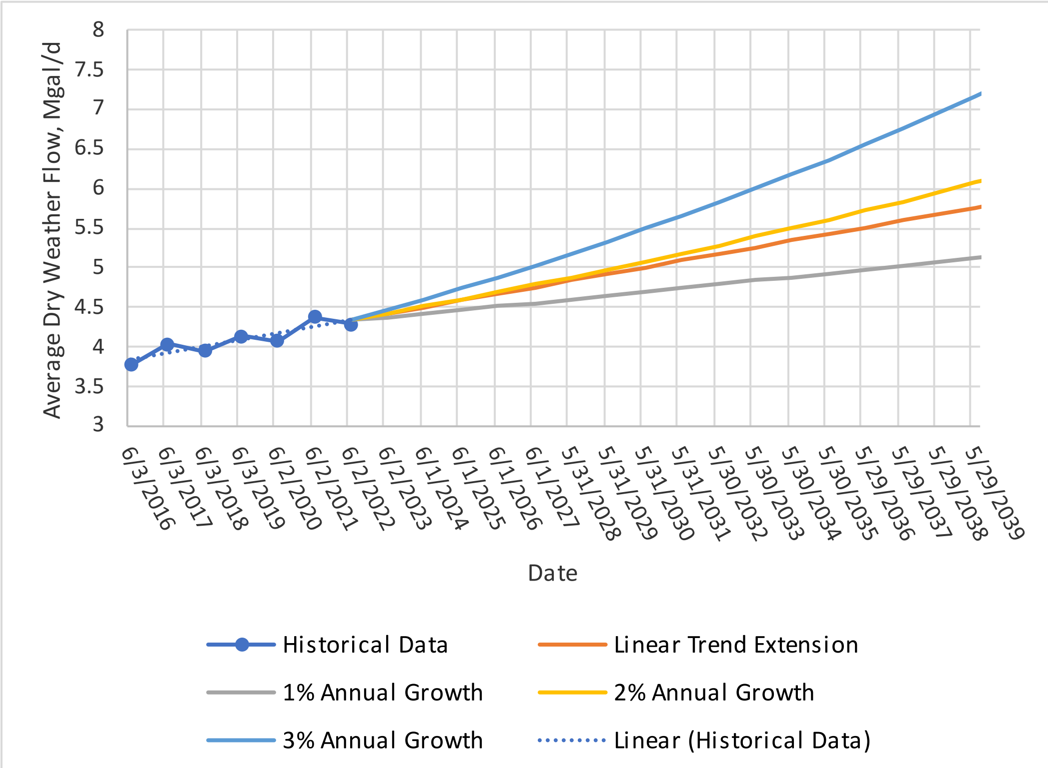

Figure 4-1 Potential Rates of Growth and Increase in Average Dry Weather Flow

4.1 MATURATION POND ANALYSIS FOR LOWER INCREMENTAL CAPACITY

Table 4-1 shows how the maturation pond equalization volume required (including safety factor and diurnal equalization storage) would vary with the design average dry weather flow and the filter system capacity. These results were derived using the same water balance procedures as previously described for the maturation ponds and would be the same for both the mainstream and sidestream alternatives, provided the maturation pond return pump capacity is adequate to prevent increased storage requirements. The diurnal storage volume was held constant at 6 Mgal, although somewhat lower values could be used for capacities less than 8 Mgal/d. As indicated, required equalization volumes increase with increased average dry weather flow and decrease with filter capacity.

Table 4-1 must be evaluated while also considering the existing maturation pond volume (177 Mgal) and the portion of that volume that can be used for equalization storage (including diurnal equalization storage). The volume that can be used for equalization storage will depend on the capacity of maturation pond outlet facilities and on the volume to be reserved for priority pollutant dilution, if any.

Table 4-1 Maturation Pond Equalization Volume Required as Determined by Design Average Dry Weather Flow and Reliable Filter Capacity

		Total Number of Filter Cells and Reliable Capacity, Mgal/d			
		6	7	8	9
		13.8	16.56	19.32	22.08
Average Dry Weather Flow, Mgal/d	5	80	40	26	19
	5.5	105	47	33	21
	6	129	67	40	27
	6.5	156	92	48	34
	7	184	116	55	41
	7.5	232	141	78	48
	8	292	167	103	55
		Body of Table is Maturation Pond Equalization Volume, Mgal			
		Volume Includes 1.25 Safety Factor and 6 Mgal Diurnal Storage Allowance			

4.1.1 Requirements for Mainstream Maturation Ponds

For the mainstream maturation pond alternative, all of the secondary effluent would be routed through the maturation ponds and the maturation pond outlet facilities (and the DAF system, unless bypassed). Therefore, these facilities must be adequate to support the full filter capacity (or additional equalization volume would be required). Currently, all effluent flow from the maturation ponds occurs by gravity with no pumping. This limits the amount of flow that can occur, particularly with decreasing pond water surface elevations needed to support increasing equalization storage requirements.

As noted previously, the maximum maturation pond water surface elevation is 112.7 ft. The maximum gravity flow capacity occurs with this maximum water surface elevation. Gravity flow is limited by adjustable weir gates in the existing Maturation Pond Level Control Structure (minimum elevation 109.1 ft) and by fixed weirs in the Dissolved Air Flotation System Splitter Box (elevation 108.08 ft). If the DAF system is bypassed, allowing maturation pond effluent to flow directly to the filters, the splitter box weirs would no longer have an impact on the maturation pond effluent flow. However, DAF bypass may not be possible in many situations, depending on the quality of the water in the maturation ponds.

If the Maturation Pond Level Control Structure is modified to remove the weir gates and lower the associated wall openings, this structure would not have a significant impact on maturation pond outflows. In this case, new control valve(s) would be needed to modulate the flow to the DAF system (or to the filters if the DAF system is bypassed). Table 4-2 shows the results of hydraulic analyses to determine the minimum maturation pond water surface elevation needed to support various filter flow capacities under the mainstream maturation pond alternative if the Maturation Pond Level Control Structure is modified as discussed. Also shown in the table is the maturation pond equalization storage volume that would be available in each scenario.

As shown in Table 4-2, gravity flow requirements severely limit available equalization storage for the mainstream maturation pond alternative. In fact, by comparing Tables 4-1 and 4-2, it can be seen that no combination of average dry weather flow and filter capacity yields an equalization storage requirement that could be met by gravity flow from the maturation ponds. Therefore, for the mainstream maturation pond alternative, without bypassing the DAF system, a new maturation pond effluent pump station is required to support any expansion of the facilities. If a new maturation pond effluent pump station is provided, it should be designed for the capacity needed for the Phase 2 design flow of 8.0 Mgal/d.

Table 4-2 Maturation Pond Water Levels and Equalization Volume vs Outlet Capacity for Mainstream Maturation Pond Alternative

Total Number of Filter Cells	Filter Capacity and Maturation Pond Outlet Gravity Flow Capacity to DAF System, Mgal/d	Minimum Maturation Pond Water Surface Elevation Required for Gravity Flow (a), ft	Maturation Pond Equalization Storage Volume Available (b), Mgal
6 (existing)	13.8 (Existing)	109.7	36
7	16.56	110.2	30
8	19.32	110.8	23
9	22.08	111.4	16

(a) Requires Maturation Pond Level Control Structure modifications (remove weir gates and lower wall openings).
(b) Volume between minimum water surface elevation needed for gravity flow and maximum water surface elevation of 112.7 ft (177 Mgal).

Although an interim capacity less than 8 Mgal/d is not reasonable for the Maturation Pond Effluent Pump Station, it is still possible to consider a lower interim capacity for the DAF, filters, UV system and other related improvements under the mainstream maturation pond alternative. This is because a new Maturation Pond Effluent Pump Station that would allow pumping down the maturation ponds to a much lower level than is currently possible with the existing gravity flow outlet system would make available much more equalization volume to accommodate more severe storm events than is currently possible. Specifically, as noted in Table 4-2, the existing gravity flow system provides for lowering the pond water surface elevation only down to 109.7 ft, resulting in 36 Mgal of available equalization storage volume with the existing filter capacity of 13.8 Mgal/d. If a new pump station was provided that would allow lowering the water surface elevation down to 101.3 (a minimum pool depth of 5 ft in the ponds, giving a minimum pool volume of 48 Mgal), the available equalization storage volume would be 177 – 48 = 129 Mgal, which is more than triple the current available volume.

As noted in Table 4-1, the volume of 129 Mgal would be adequate to accommodate a plant capacity of 5, 5.5, or 6.0 Mgal/d without expanding the filter capacity. However, for the 6.0 Mgal/d capacity, the equalization storage volume available is the same as the recommended volume (129 Mgal). Although the recommended volume does include a 1.25 safety factor, this should still be considered marginal. To provide additional reliability and operational flexibility, including the ability to handle more severe storm events than those occurring in December 2022 and January 2023, increasing the filter capacity to at least

16.56 Mgal/d by adding one additional filter cell may be prudent for a capacity of 6 Mgal/d. Furthermore, since a minimum filter capacity of 19.32 Mgal/d would be required for the subsequent expansion to 8 Mgal/d, it may make sense to provide that capacity even for a 6 Mgal/d project, thereby avoiding expanding the filters twice with only perhaps 5 years from end of construction of the first expansion to the beginning of construction of the second expansion. This would increase the up-front costs but would decrease the overall cost of providing two additional filter cells.

It must be recognized that reducing the minimum pool volume to 48 Mgal also reduces the hydraulic retention time for priority pollutant dilution. At a peak flow of 16.56 Mgal/d (one filter cell added), the retention time would be 2.9 days. At 6.6 Mgal/d (10% recycle allowance above 6 Mgal/d ADWF), the retention time would be 7.3 days.

The various improvements that would be required for an interim capacity of 6 Mgal/d are shown in Table 4-1 presented later in this document.

4.1.2 Requirements for Sidestream Maturation Ponds

When the sidestream maturation pond alternative is considered, it is possible to consider a much lower capacity for the maturation pond effluent pump system. This is because, during maturation pond drawdown, the maturation pond effluent flow rate is not the desired filter flow (as it is for mainstream ponds), rather, the desired filter flow minus the secondary effluent flow. Furthermore, as developed in Figure 2-7, this flow can be reduced significantly without impacting the maximum maturation pond equalization storage requirement.

Analyses such as used to develop Figure 2-7 were completed for average dry weather flows ranging from 5.0 to 6.0 Mgal/d and for filter capacities ranging from 13.8 to 19.32 Mgal/d (six filter cells [existing] to eight filter cells). Recommended minimum capacities for the maturation pond effluent pump system resulting from those analyses are indicated in Table 4-3. The pump capacities shown represent average daily pumping rates plus a diurnal flow allowance of 75 percent of the average dry weather flow. The values shown in the table are minimums, while additional operational flexibility would be available with higher capacities. The final capacities should be determined based on what is reasonably possible with minor modifications to existing facilities, which is discussed further below.

Table 4-3 Recommended Sidestream Maturation Pond Return Pumping Capacities

	Top Row is Filter Capacity, Mgal/ d		
ADWF, Mgal/ d	13.8	16.56	19.32
5.0	7.8	7.3	6.8
5.5	8.1	7.6	7.1
6.0	8.5	8.0	7.5
	Body of Table is Recomended Mat Pond Ret Flow, Mgal/ d Including Diurnal Allowance = 75% of ADWF		

Since the maturation pond effluent pumping requirements for a capacity of 6 Mgal/d ADWF are reasonably attainable (as discussed below), the analysis of maturation pond storage requirements and water levels discussed below are based on this capacity.

In Table 4-4, minimum maturation pond equalization storage requirements and the associated maturation pond minimum water levels are shown for the average dry weather flow capacity of 6 Mgal/d and for various filter capacities. Estimated maturation pond return pump static heads are shown also. As was noted for the mainstream alternative and as shown for the sidestream alternative in Table 4-4, a plant capacity of 6 Mgal/d (ADWF) can be accommodated for the sidestream alternative with the existing filter capacity of 13.8 Mgal/d. However, as previously discussed for the mainstream alternative, it may be prudent to provide a filter capacity of 16.56 or 19.32 Mgal/d for a plant capacity of 6 Mgal/d (ADWF).

For the sidestream alternative, as for the mainstream alternative, a maturation pond minimum pool volume of 48 Mgal at a depth of 5 feet is required for expansion to 6 Mgal/d (ADWF) with a filter capacity of 13.8 Mgal/d. Although higher minimum pool volumes and water surface elevations are possible for higher filter capacities, it may be desirable to use the same low minimum pool for all filter capacities, as this would maximize operational flexibility and minimize hydraulic residence times, thereby minimizing algae growth.

Based on a hydraulic analysis, the existing maturation pond outlet pumps should be able to produce about 3.85 Mgal/d each (total of 7.7 Mgal/d) down to a minimum pool volume of 48 Mgal/d at a maturation pond residual depth of 5 feet. However, if the last 40 feet of piping, which is currently combined for both pump discharges, is revised with parallel pipes, the total flow could be increased to about 9.1 Mgal/d, which would exceed the requirements shown in Table 4-3 for all plant and filter capacities considered. Existing pump performance should be verified by field testing.

Table 4-4 Sidestream Maturation Pond Equalization Storage Requirements, Water Levels, and Maturation Pond Return Pump Static Heads, for 6 Mgal/d Average Dry Weather Flow

Total Number of Filter Cells	Filter System Reliable Capacity, Mgal/d	Minimum Maturation Pond Equalization Storage Requirement (a), Mgal	Maximum Maturation Pond Residual Volume When Minimum Equalization Storage Volume is Empty (b), Mgal	Maturation Pond Water Surface Elevation When Minimum Equalization Storage Volume is Empty, ft	Maturation Pond Depth at Minimum Water Surface Elevation (d), ft	Minimum Design Static Head for Return Pump, ft
6 (existing)	13.8 (Existing)	129	48	101.3	5.0	11.2
7	16.56	67	110	107.1	10.8	5.4
8	19.32	40	137	109.4	13.1	3.1

(a) From Table 4-1.
(b) Total volume of 177 Mgal minus equalization volume.
(c) See Figure 1-1.
(d) Based on average pond bottom elevation of 96.3.
(e) Based on assumed discharge centerline elevation of 112.5 at Maturation Pond Level Control Structure.

To summarize the information provided above, an initial expansion capacity of 6 Mgal/d can be considered for the maturation pond sidestream alternative without expansion of the existing filter system. However, filter system expansion by adding one or two additional filter cells may be prudent. The existing maturation pond effluent pumps, with minor piping modifications, should be able to produce up to 9.1 Mgal/d, which exceeds the minimum requirement for all plant and filter capacities considered at the recommended minimum depth of 5 feet in the maturation ponds (minimum pool volume = 48 Mgal).

4.2 TERTIARY STORAGE BASIN ANALYSIS FOR LOWER INCREMENTAL CAPACITY

As developed previously, even at the design capacity of 8 Mgal/d, no expansion of the tertiary storage basins is needed. Therefore, it is not necessary to consider lower design capacities.

5.0 UPDATED CONSIDERATIONS AND RECOMMENDATIONS REGARDING THE OPERATION OF AND RECOMMENDED IMPROVEMENTS TO THE MATURATION POND FACILITIES, TERTIARY STORAGE BASIN FACILITIES, AND OTHER PLANT FACILITIES IMPACTED BY THESE CONSIDERATIONS

A summary of considerations and facilities requirements developed in the previous sections is presented in Table 5-1.

Table 5-1 Summary of Considerations and Facilities Requirements with Mainstream and Sidestream Maturation Ponds

Facility or Consideration	Design Capacity 8.0 Mgal/d Average Dry Weather Flow		Design Capacity 6.0 Mgal/d Average Dry Weather Flow	
	Mainstream Maturation Ponds	Sidestream Maturation Ponds	Mainstream Maturation Ponds	Sidestream Maturation Ponds
Priority Pollutant Dilution in Maturation Ponds	Can be operated with various levels of dilution, dependent on minimum water level and volume reserved for flow equalization. With 129 Mgal reserved for equalization storage, a volume of 48 Mgal would be available for priority pollutant dilution, yielding a hydraulic residence time of 5.5 days with a future dry weather flow of 8.8 Mgal/d (includes 10% recycle allowance).	Substantial priority pollutant dilution not provided.	Can be operated with various levels of dilution, dependent on minimum water level and volume reserved for flow equalization. With 129 Mgal reserved for equalization storage, a volume of 48 Mgal would be available for priority pollutant dilution, yielding a hydraulic residence time of 7.3 days with a future dry weather flow of 6.6 Mgal/d (includes 10% recycle allowance).	Substantial priority pollutant dilution not provided.
Effluent Cooling in Maturation Ponds to Aid in Temperature Compliance	Substantial cooling provided to assure easier compliance with daily and annual average temperature limitations.	Minimal cooling provided. Should still comply with permit temperature requirements, but with less margin of safety as compared to the mainstream alternative.	Substantial cooling provided to assure easier compliance with daily and annual average temperature limitations.	Minimal cooling provided. Should still comply with permit temperature requirements, but with less margin of safety as compared to the mainstream alternative.
Natural Disinfection in Maturation Ponds	Substantial disinfection providing, easing requirements for UV disinfection.	Minimal disinfection provided. Higher UV disinfection system dose requirements compared to the mainstream alternative.	Substantial disinfection providing, easing requirements for UV disinfection.	Minimal disinfection provided. Higher UV disinfection system dose requirements compared to the mainstream alternative.
Secondary Process Backup Provided	Yes	Mostly no.	Yes	Mostly no.
Diurnal Equalization of Flow to DAF, Filters, and UV.	Easily provided by regulating outflow from maturation ponds.	Complex, requiring coordinated control of four flow rates, involving three pump systems and flow recycling between the maturation ponds and DAF.	Easily provided by regulating outflow from maturation ponds.	Complex, requiring coordinated control of four flow rates, involving three pump systems and flow recycling between the maturation ponds and DAF.
Maturation Pond Feed Pump Station Capacity Required, Mga/d	50.0 (compare to existing capacity of 33.1 Mgal/d) (a)	50.0 minus filter capacity, e.g., 30.7 Mgal/d with 8 filter cells. (a)	41.0 Mgal/d (compare to existing capacity of 33.1 Mgal/d) (a)	Approximately 22 Mgal/d. Existing capacity of 33.1 Mgal/d exceeds requirements. No expansion required. (a)
Maturation Ponds	With eight filter cells, the recommended minimum equalization storage volume (including safety and diurnal equalization allowances) is 103 Mgal. The available equalization volume would increase to about 129 Mgal, based on maintaining a minimum pool depth of 5 feet in the maturation ponds..	With eight filter cells, the recommended minimum equalization storage volume (including safety and diurnal equalization allowances) is 103 Mgal. The available equalization volume would increase to about 129 Mgal, based on maintaining a minimum pool depth of 5 feet in the maturation ponds..	With seven and eight filter cells, the recommended minimum equalization storage volumes (including safety and diurnal equalization allowances) are 67 and 40 Mgal, respectively. Flexibility to lower the maturation pond level to a depth of 5 feet would result in an equalization volume of 129 Mgal.	With seven and eight filter cells, the recommended minimum equalization storage volumes (including safety and diurnal equalization allowances) are 67 and 40 Mgal, respectively. Flexibility to lower the maturation pond level to a depth of 5 feet would result in an equalization volume of 129 Mgal.

Facility or Consideration	Design Capacity 8.0 Mgal/d Average Dry Weather Flow		Design Capacity 6.0 Mgal/d Average Dry Weather Flow	
	Mainstream Maturation Ponds	Sidestream Maturation Ponds	Mainstream Maturation Ponds	Sidestream Maturation Ponds
Maturation Pond Effluent Pump Station	With eight filter cells, the required capacity is 19.32 Mgal/d. Based on an equalization volume of 103 Mgal, the pumps must be capable of pumping the required capacity at a maturation pond water surface elevation of about 103.8 ft. Additional flexibility would be provided by the ability to pump the maturation ponds down to a water surface elevation of 101.3 ft. (Compare to completed Phase 1 design for 15.3 Mgal/d down to water surface elevation 105.8 ft.)	With eight filter cells, the recommended capacity is 10 Mgal/d. Based on an equalization volume of 103 Mgal, the pumps must be capable of pumping the required capacity at a maturation pond water surface elevation of about 103.8 ft. Additional flexibility would be provided by the ability to pump the maturation ponds down to a water surface elevation of 101.3 ft. The existing pond effluent pump system can likely meet these requirements with minor modifications.	Design for 8 Mgal/d ADWF condition. See first column this table.	With minor piping modifications, the existing maturation pond drain pump system should be able to provide a capacity of 9.1 Mgal/d with a minimum maturation pond water surface elevation of 101.3 ft. This exceeds the minimum requirement for any filter capacity considered. Coordinate with DAF capacity below.
Dissolved Air Flotation System	At capacity of 8 Mgal/d each, 3 DAF clarifiers would be needed to handle the entire filter feed flow if 19.32 Mgal/d. Partial DAF overload or bypass could be considered to allow only 2 DAF clarifiers. One DAF clarifier for maturation pond use is currently existing. A second existing DAF clarifier is currently used only for TSB return flows but could be considered for maturation pond use also. For redundancy, a third DAF may be desired.	The recommended maturation pond return flow of 10 Mgal/d would exceed the capacity of one DAF clarifier (8 Mgal/d). The recommended return flow of 10 Mgal/d would require a second DAF or overload or partial bypass. For redundancy, a third DAF may be desired.	At capacity of 8 Mgal/d each, 2 DAF clarifiers would be adequate to handle the flow for 7 filters, if the filter flow is reduced slightly from the maximum capacity of 16.56 Mgal/d to 16.0 Mgal/d Three DAF clarifiers would be needed to handle the full capacity of 19.32 Mgal/d for 8 filters, unless partial DAF overload or bypass is considered to allow only 2 DAF clarifiers. One DAF clarifier for maturation pond use is currently existing. A second existing DAF clarifier is currently used only for TSB return flows but could be considered for maturation pond use also. For redundancy, a third DAF may be desired.	Existing DAF capacity of 8 Mgal/d is only slightly lower than the recommended minimum maturation pond return flow of 8.5 Mgal/d for a filter capacity of 13.8 Mgal/d but reducing the return flow to 8.0 Mgal/d would be reasonable. The existing DAF capacity meets or exceeds the minimum recommended maturation pond return flows for filter capacities of 16.56 and 19.32 Mgal/d (8.0 and 7.5 Mgal/d). If flexibility for a higher flow of 9.1 Mgal/d is provided, partial DAF overload or bypass would be required. A second existing DAF clarifier is currently used only for TSB return flows but could be considered for maturation pond use also to provide redundancy. No DAF expansion recommended.
Filters and Filter Feed Pump Station Capacity, Mgal/d	With eight filter cells, the capacity would be 19.32 Mgal/d.	With eight filter cells, the capacity would be 19.32 Mgal/d.	With seven filter cells, the capacity would be 16.56 Mgal/d. With eight filter cells, the capacity would be 19.32 Mgal/d.	With seven filter cells, the capacity would be 16.56 Mgal/d. With eight filter cells, the capacity would be 19.32 Mgal/d.

Facility or Consideration	**Design Capacity 8.0 Mgal/d Average Dry Weather Flow**		**Design Capacity 6.0 Mgal/d Average Dry Weather Flow**	
	Mainstream Maturation Ponds	**Sidestream Maturation Ponds**	**Mainstream Maturation Ponds**	**Sidestream Maturation Ponds**
UV Disinfection System	Current UV capacity is 17.5 Mgal/d. Adding lamps to an existing empty channel would increase capacity to 21 Mgal/d. This would be adequate for the capacity of eight filter cells (19.32 Mgal/d). Increasing UV capacity to 22.08 Mgal/d to match the capacity of nine filter cells would require a more substantial expansion. Alternatively, a 21 Mgal/d UV capacity could accommodate nine filter cells operated at less than full capacity (21 vs 22.08 Mgal/d).	Current UV capacity is 17.5 Mgal/d. Adding lamps to an existing empty channel would increase capacity to 21 Mgal/d. This would be adequate for the capacity of eight filter cells (19.32 Mgal/d). Increasing UV capacity to 22.08 Mgal/d to match the capacity of nine filter cells would require a more substantial expansion. Alternatively, a 21 Mgal/d UV capacity could accommodate nine filter cells operated at less than full capacity (21 vs 22.08 Mgal/d).	Current UV capacity of 17.5 Mgal/d is adequate for the full capacity of seven filter cells (16.56 Mgal/d). Adding lamps to an existing empty channel would increase capacity to 21 Mgal/d. This would be adequate for the capacity of eight filter cells (19.32 Mgal/d).	Current UV capacity of 17.5 Mgal/d is adequate for the full capacity of seven filter cells (16.56 Mgal/d). Adding lamps to an existing empty channel would increase capacity to 21 Mgal/d. This would be adequate for the capacity of eight filter cells (19.32 Mgal/d).
Effluent Pump Station	25 Mgal/d required to maximize discharge when temperature and flow conditions permit, thereby minimizing diversions to the tertiary storage basins, but this is not needed because tertiary storage basins have surplus capacity. Existing Effluent Pump Station capacity of 20.4 Mgal/d is adequate.	25 Mgal/d required to maximize discharge when temperature and flow conditions permit, thereby minimizing diversions to the tertiary storage basins, but this is not needed because tertiary storage basins have surplus capacity. Existing Effluent Pump Station capacity of 20.4 Mgal/d is adequate.	25 Mgal/d required to maximize discharge when temperature and flow conditions permit, thereby minimizing diversions to the tertiary storage basins, but this is not needed because tertiary storage basins have surplus capacity. Existing Effluent Pump Station capacity of 20.4 Mgal/d is adequate.	25 Mgal/d required to maximize discharge when temperature and flow conditions permit, thereby minimizing diversions to the tertiary storage basins, but this is not needed because tertiary storage basins have surplus capacity. Existing Effluent Pump Station capacity of 20.4 Mgal/d is adequate.
Tertiary Storage Basins Capacity Required	Likely no storage required for temperature compliance. Modest storage (perhaps 50 Mgal) required for irrigation operations. Existing storage capacity is 190 Mgal. No expansion of existing basins needed.	At least 98 Mgal (without safety factor) required for temperature compliance based on available data. A substantial safety factor is warranted. Existing storage capacity is 190 Mgal. No expansion of existing basins needed.	Likely no storage required for temperature compliance. Modest storage (perhaps 50 Mgal) required for irrigation operations. Existing storage capacity is 190 Mgal. No expansion of existing basins needed.	Not specifically analyzed. No capacity expansion required.

(a) A peak hour plant influent flow of 31.3 Mgal/d was experienced on January 10, 2017 (28.0 was experienced on December 31, 2022). The currently projected future peak hour influent flows resulting from the historical flows are 50 Mgal/d and 41 Mgal/d, corresponding to design average dry weather flows of 8 and 6 Mgal/d, respectively. It is beyond the scope of this study to determine how such high peak flows would be handled by plant facilities from the influent pump station and headworks through the secondary process (secondary process evaluations are currently being developed separate from this study). The Maturation Pond Feed Pump Station flows listed in this table are place-holder values that match the projected influent flows and do not take into account plant recycle flows or rainfall captured on the plant site or consideration of possible diversions to the emergency storage basins. These issues must be investigated before plant expansion design.

As developed in this study and summarized in Table 5-1 for expansion to 8 Mgal/d ADWF, the sidestream maturation pond alternative would allow smaller capacities for the maturation pond effluent pumping and DAF systems. However, those benefits are offset by considerable negative impacts regarding effluent cooling, effluent disinfection, secondary process backup, priority pollutant dilution, and complex tertiary process flow controls. Therefore, mainstream maturation ponds are recommended for expansion to 8 Mgal/d.

Both mainstream and sidestream maturation ponds could be considered for an interim 6 Mgal/d ADWF expansion, if it is desired to minimize near-term costs. With the mainstream alternative, a new maturation pond effluent pump station suitable for the future 8 Mgal/d capacity would be required from the outset, but savings could be realized by sizing DAF, filter, and UV systems for 6 Mgal/d instead of 8 Mgal/d. For the sidestream alternative, the interim project would be much less expensive because a new maturation pond effluent pump station would not be required, and DAF capacity could be reduced as compared to the mainstream alternative. However, the negative aspects of sidestream maturation ponds would still be applicable at the reduced capacity. The most robust solution is to continue to use the mainstream maturation pond configuration for interim and future expansions.

When considering mainstream versus sidestream maturation ponds for either 6 Mgal/d or 8 Mgal/d (or any other capacity), it must be recognized that the secondary process backup that is provided by the mainstream configuration but is not provided by the sidestream configuration has major implications for secondary process design and cost. Therefore, the selection of a maturation pond alternative must be coordinated with secondary process evaluations that are the subject of a separate investigation.

Based on the above findings and knowledge of community growth rates and budgets, a capacity of 6 Mgal/d ADWF is recommended for the next WWTRF expansion. Continuing the current configuration of mainstream maturation ponds is also recommended. However, some costs for the facilities considered in this study can be deferred by switching to the sidestream maturation pond configuration (if reasonable after coordination with secondary process evaluations). Table 5-2 summarizes the treatment facilities needed for tertiary treatment at 6 Mga/d ADWF compared to the treatment facilities included in the completed 8 Mgal/d ADWF design. As developed in this study and based on new information, the completed design would have to be modified to attain the 8 Mgal/d capacity.

Table 5-2 Summary of Treatment Facilities Needed at 6 Mgal/d ADWF Compared to Completed Design

Facility	Mainstream	Sidestream	Current Design
Maturation Pond Feed Pump Station	Expand to 41.0 Mgal/d (a)	No expansion required	Not expanded
Maturation Pond Effluent Pump Station	New pump station required with capacity of 19.32 or 22.08 Mgal/d (depending on filter capacity) capable of pumping down to maturation pond water surface elevation of 103.8 ft or, for more flexibility, 101.3 ft.	Minor modification to existing piping	New pump station with capacity of 15.3 Mga/d capable of pumping down to maturation pond water surface elevation of 105.8 ft.
Dissolved Air Flotation	Interconnect existing DAF systems and add one new DAF	Interconnect existing DAF systems	One new DAF
Filters and Filter Feed Pump Station	Add one filter cell and one feed pump. Can consider adding two filter cells and one feed pump and replacing another feed pump.	Add one filter cell and one feed pump. Can consider adding two filter cells and one feed pump and replacing another feed pump.	One filter cell and one feed pump
UV Disinfection	Expansion not required, but recommended based on operational best practice if only one filter cell is added. Expansion required to match the capacity of two filter cells added.	Expansion not required, but recommended based on operational best practice if only one filter cell is added. Expansion required to match the capacity of two filter cells added.	Equip empty channel with new UV lamps

(a) See footnote (a) under Table 5-1.

APPENDIX A

WATER BALANCES

City of Lincoln WWTRF

WATER BALANCE - PROJECTED 8.0 MGD ADWF, WATER YR 2019 ALLOWABLE DISCHARGES BASED ON 15 MIN CALCS, WITH 2023 PERMIT REVISIONS WITH 1F SAFETY MARGIN
OXIDATION DITCH TEMPERATURES USED

13-Apr-23
3:12 PM

OVERALL INPUT DATA

FLOWS AND INFILTRATION/INFLOWS (I/I)		CLIMATOLOGICAL AND RUNOFF FACTORS	
ADWF (MGD)..	8.00	OCT-APR EVAP/AVG EVAP RATIO..................	1.00
		MAY-SEP EVAP/AVG EVAP RATIO..................	1.00
		PAN COEFFICIENT..	0.80

PLANT SITE, MATURATION POND, AND TERTIARY STORAGE BASIN INPUT	MAT POND*	TERT STOR
RAIN CATCH AREA (AC) (*MAT POND + PLANT SITE).....................	95.0	46.2
MIN WATER SURFACE AREA (AC)..	40.0	35.4
MAX WATER SURFACE AREA (AC)..	40.0	41.4
MAX TERTIARY EFFL STORAGE (MG)...	---	190.0
LAND PRECIP COLLECTED (FRAC)...	0.90	0.90

IRRIGATION INPUT DATA	AGRICULTURE	LANDSCAPE
IRRIGATION AREA (AC)...	942.0	0.0
IRRIGATION EFFICIENCY (FRACTION)...	0.700	0.700
SOIL WATER DEFICIT BEFORE IRRIG. (IN)......................................	1.0	1.0

MONTHLY INPUT DATA

	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
DAYS IN MONTH	31	30	31	31	28	31	30	31	30	31	31	30	365
AVG PAN EVAP (IN)	4.89	2.06	1.25	0.92	1.90	3.47	5.21	8.07	9.91	11.12	9.93	7.45	66.18
WATER YEAR 2019 PRECIP (IN)	0.23	1.85	1.10	4.54	6.87	2.98	0.55	2.57	0.00	0.00	0.00	0.66	21.35
WATER YEAR 2019 Eto (IN)	4.19	2.16	1.03	0.98	1.11	2.80	5.17	5.53	7.94	8.33	7.61	5.51	52.38
AGRICULTURE CROP COEFF (ALFALFA)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
LANDSCAPE CROP COEFF (GRASS)	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	
INDUSTRIAL DEMAND (MGD)	0	0	0	0	0	0	0	0	0	0	0	0	
WYR 2019 MAX ALLOWABLE DISCH TO CREEK (MGD)	3.37	9.82	14.18	12.29	15.80	12.55	8.73	9.49	0.00	0.00	0.00	8.24	
WYR 2019 INFILTRATION AND INFLOW (I/I) (MGD)	0.07	0.91	1.96	3.71	6.84	4.21	1.42	1.48	0.28	0.02	0.07	0.16	

CALCULATIONS

	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
INFLUENT INCLUDING I/I (MGD)	8.07	8.91	9.96	11.71	14.84	12.21	9.42	9.48	8.28	8.02	8.07	8.16	
EVAPORATION FROM PONDS (IN)	3.9	1.6	1.0	0.7	1.5	2.8	4.2	6.5	7.9	8.9	7.9	6.0	52.9
MATURATION POND													
INFLOW (MG)	250.09	267.45	308.74	362.98	415.47	378.41	282.57	293.85	248.54	248.59	250.21	244.72	3551.6
PRECIP. VOLUME (MG)	0.56	4.50	2.68	11.04	16.71	7.25	1.34	6.25	0.00	0.00	0.00	1.61	51.9
EVAP. VOLUME (MG)	4.25	1.79	1.09	0.80	1.65	3.02	4.53	7.02	8.62	9.67	8.63	6.48	57.5
OUTFLOW (MG)	246.39	270.15	310.32	373.23	430.52	382.64	279.38	293.09	239.92	238.92	241.57	239.84	3546.0
OUTFLOW (MGD)	7.95	9.01	10.01	12.04	15.38	12.34	9.31	9.45	8.00	7.71	7.79	7.99	
AGRICULTURE IRRIGATION													
EVAPOTRANSPIRATION (IN)	4.19	2.16	1.03	0.98	1.11	2.80	5.17	5.53	7.94	8.33	7.61	5.51	52.4
IRRIG DEMAND = ET-PRECIP (IN)	3.96	0.31	0.00	0.00	0.00	0.00	4.62	2.96	7.94	8.33	7.61	4.85	40.6
REDUCTION FOR DEFICIT (IN)	0.00	0.00	-0.07	-0.93	0.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00
CUM RED FOR DEFICIT (IN)	1.00	1.00	0.93	0.00	0.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	
DEFICIT NOT SATISFIED (IN)	0.00	0.0	0.1	1.0	1.0	1.0	0.0	0.0	0.0	0.0	0.0	0.0	
REVISED IRRIG DEMAND (IN)	3.96	0.31	0.00	0.00	0.00	0.00	3.62	2.96	7.94	8.33	7.61	4.85	39.58
REVISED IRRIGATION DEMAND (MG)	144.9	11.5	0.0	0.0	0.0	0.0	132.4	108.4	290.4	304.6	278.2	177.2	1447.5
REVISED IRRIGATION DEMAND (MGD)	4.68	0.38	0.00	0.00	0.00	0.00	4.41	3.50	9.68	9.83	8.97	5.91	
LANDSCAPE IRRIGATION													
EVAPOTRANSPIRATION (IN)	2.94	1.51	0.72	0.69	0.78	1.96	3.62	3.87	5.56	5.83	5.32	3.85	36.7
IRRIG DEMAND = ET-PRECIP (IN)	2.71	0.00	0.00	0.00	0.00	0.00	3.07	1.30	5.56	5.83	5.32	3.19	27.0
REDUCTION FOR DEFICIT (IN)	0.00	-0.34	-0.38	-0.29	0.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00
CUM RED FOR DEFICIT (IN)	1.00	0.66	0.29	0.00	0.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	
DEFICIT NOT SATISFIED (IN)	0.00	0.3	0.7	1.0	1.0	1.0	0.0	0.0	0.0	0.0	0.0	0.0	
REVISED IRRIG DEMAND (IN)	2.71	0.00	0.00	0.00	0.00	0.00	2.07	1.30	5.56	5.83	5.32	3.19	25.99
REVISED IRRIGATION DEMAND (MG)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
REVISED IRRIGATION DEMAND (MGD)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
INDUSTRIAL DEMAND (MG)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
EFFLUENT ROUTING ANALYSIS													
MAXIMUM POSSIBLE DISCHARGE TO CREEK (MG)	104.48	294.64	439.64	380.87	442.52	388.94	261.85	294.17	0.00	0.00	0.00	247.32	2854.43
TOTAL REUSE DEMAND (MG)	144.93	11.46	0.00	0.00	0.00	0.00	132.38	108.36	290.35	304.61	278.16	177.23	1447.50
MAXIMUM POSSIBLE DISCHARGE + REUSE (MG)	249.41	306.10	439.64	380.87	442.52	388.94	394.23	402.53	290.35	304.61	278.16	424.55	
VOLUME AVAILABLE FOR DISCHARGE + REUSE (MG)	246.39	270.15	311.01	373.61	435.38	389.60	281.00	293.09	239.92	238.92	241.57	239.84	
ACTUAL REUSE (MG)	144.93	11.46	0.00	0.00	0.00	0.00	132.38	108.36	239.92	238.92	241.57	177.23	1294.78
ACTUAL REUSE (MGD)	4.68	0.38	0.00	0.00	0.00	0.00	4.41	3.50	8.00	7.71	7.79	5.91	
REUSE DEMAND NOT SATISFIED (MG)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	50.43	65.69	36.59	0.00	152.72
REUSE DEMAND NOT SATISFIED (MGD)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.68	2.12	1.18	0.00	
ACTUAL DISCHARGE TO CREEK (MG)	101.46	258.70	311.01	373.61	435.38	388.94	148.62	184.72	0.00	0.00	0.00	62.61	2265.05
ACTUAL DISCHARGE TO CREEK (MGD)	3.27	8.62	10.03	12.05	15.55	12.55	4.95	5.96	0.00	0.00	0.00	2.09	
UNUSED DISCHARGE CAPACITY (MG)	3.02	35.95	128.64	7.25	7.14	0.00	113.23	109.44	0.00	0.00	0.00	184.71	589.38
UNUSED DISCHARGE CAPACITY (MGD)	0.10	1.20	4.15	0.23	0.26	0.00	3.77	3.53	0.00	0.00	0.00	6.16	
TSB OUTFLOW - TSB INFLOW (MG)	0.00	0.00	0.68	0.39	4.86	6.30	1.62	0.00	0.00	0.00	0.00	0.00	
TERTIARY STORAGE BASINS													
BEGINNING STORAGE (MG)	0.00	0.00	0.68	0.39	4.86	6.96	1.62	0.00	0.00	0.00	0.00	0.00	
BEGINNING WATER SURFACE AREA (AC)	35.40	35.40	35.42	35.41	35.55	35.62	35.45	35.40	35.40	35.40	35.40	35.40	
EVAP. VOLUME (MG)	0.28	1.59	0.96	0.71	1.47	2.69	0.67	3.15	0.00	0.00	0.00	0.81	12.33
PRECIP. VOLUME (MG)	0.28	2.27	1.35	5.57	8.43	3.66	0.67	3.15	0.00	0.00	0.00	0.81	26.18
STORAGE GAIN (MG)	0.00	0.68	-0.30	4.47	2.10	-5.34	-1.62	0.00	0.00	0.00	0.00	0.00	0.00
FINAL STORAGE (MG)	0.00	0.68	0.39	4.86	6.96	1.62	0.00	0.00	0.00	0.00	0.00	0.00	

SUMMARY

ANNUAL INFLOW (MG)	
WASTEWATER WITHOUT I/I	2920
INFLOW AND INFILTRATION..	632
PRECIP. INTO PONDS/BASINS..	78
TOTAL..	3630

ANNUAL OUTFLOW (MG)	
DISCHARGE TO STREAM...	2265
TOTAL ALL REUSE..	1295
EVAP. FROM PONDS/BASINS..	70
TOTAL..	3630

INFLOW-OUTFLOW AND STORAGE (MG)	
ANNUAL INFLOW - ANNUAL OUTFLOW (MG)	**0**
STORAGE AVAILABLE (MG)...	190
STORAGE REQUIRED (MG)..	7
SURPLUS STORAGE CAPACITY (MG)..	**183**

CREEK DISCHARGE AND REUSE SUMMARY	
MAXIMUM POSSIBLE CREEK DISCHARGE (MG)	2854
ACTUAL CREEK DISCHARGE (MG)..	2265
UNUSED CREEK DISCHARGE CAPACITY (MG).............................	**589**
REUSE DEMAND (MG)..	1447
ACTUAL REUSE (MG)...	1295
REUSE DEMAND NOT SATISFIED (MG)...	**153**

City of Lincoln WWTRF

WATER BALANCE - PROJECTED 8.0 MGD ADWF, WATER YR 2020 ALLOWABLE DISCHARGES BASED ON 15 MIN CALCS, WITH 2023 PERMIT REVISIONS WITH 1F SAFETY MARGIN
OXIDATION DITCH TEMPERATURES USED

13-Apr-23
3:11 PM

OVERALL INPUT DATA

FLOWS AND INFILTRATION/INFLOWS (I/I)	
ADWF (MGD)..	8.00

CLIMATOLOGICAL AND RUNOFF FACTORS	
OCT-APR EVAP/AVG EVAP RATIO..................	1.00
MAY-SEP EVAP/AVG EVAP RATIO..................	1.00
PAN COEFFICIENT...	0.80

PLANT SITE, MATURATION POND, AND TERTIARY STORAGE BASIN INPUT	MAT POND*	TERT STOR
RAIN CATCH AREA (AC) (*MAT POND + PLANT SITE).....................	95.0	46.2
MIN WATER SURFACE AREA (AC)..	40.0	35.4
MAX WATER SURFACE AREA (AC)..	40.0	41.4
MAX TERTIARY EFFL STORAGE (MG)...	---	190.0
LAND PRECIP COLLECTED (FRAC)..	0.90	0.90

IRRIGATION INPUT DATA	AGRICULTURE	LANDSCAPE
IRRIGATION AREA (AC)...	942.0	0.0
IRRIGATION EFFICIENCY (FRACTION)...	0.700	0.700
SOIL WATER DEFICIT BEFORE IRRIG. (IN).......................................	1.0	1.0

MONTHLY INPUT DATA

	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
DAYS IN MONTH	31	30	31	31	28	31	30	31	30	31	31	30	365
AVG PAN EVAP (IN)	4.89	2.06	1.25	0.92	1.90	3.47	5.21	8.07	9.91	11.12	9.93	7.45	66.18
WATER YEAR 2020 PRECIP (IN)	0.00	0.61	5.34	1.23	0.00	1.75	1.31	0.22	0.11	0.00	0.03	0.00	10.60
WATER YEAR 2020 Eto (IN)	4.60	2.34	0.93	1.22	3.21	3.26	5.03	6.76	8.09	8.48	7.23	5.36	56.50
AGRICULTURE CROP COEFF (ALFALFA)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
LANDSCAPE CROP COEFF (GRASS)	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	
INDUSTRIAL DEMAND (MGD)	0	0	0	0	0	0	0	0	0	0	0	0	
WYR 2020 MAX ALLOWABLE DISCH TO CREEK (MGD)	4.54	6.55	16.48	9.24	8.26	9.40	8.88	9.43	0.00	0.00	0.00	8.00	
WYR 2020 INFILTRATION AND INFLOW (I/I) (MGD)	0.03	0.14	2.69	1.08	0.34	1.24	1.60	0.49	0.14	0.00	0.00	0.00	

CALCULATIONS

	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
INFLUENT INCLUDING I/I (MGD)	8.03	8.14	10.69	9.08	8.34	9.24	9.60	8.49	8.14	8.00	8.00	8.00	
EVAPORATION FROM PONDS (IN)	3.9	1.6	1.0	0.7	1.5	2.8	4.2	6.5	7.9	8.9	7.9	6.0	52.9
MATURATION POND													
INFLOW (MG)	248.97	244.08	331.53	281.55	233.55	286.47	287.88	263.29	244.34	248.08	248.04	240.00	3157.8
PRECIP. VOLUME (MG)	0.00	1.48	12.99	2.99	0.00	4.26	3.19	0.54	0.27	0.00	0.07	0.00	25.8
EVAP. VOLUME (MG)	4.25	1.79	1.09	0.80	1.65	3.02	4.53	7.02	8.62	9.67	8.63	6.48	57.5
OUTFLOW (MG)	244.72	243.78	343.43	283.74	231.90	287.71	286.54	256.81	235.99	238.41	239.48	233.52	3126.0
OUTFLOW (MGD)	7.89	8.13	11.08	9.15	8.28	9.28	9.55	8.28	7.87	7.69	7.73	7.78	
AGRICULTURE IRRIGATION													
EVAPOTRANSPIRATION (IN)	4.60	2.34	0.93	1.22	3.21	3.26	5.03	6.76	8.09	8.48	7.23	5.36	56.5
IRRIG DEMAND = ET-PRECIP (IN)	4.60	1.73	0.00	0.00	3.21	1.51	3.72	6.54	7.98	8.48	7.20	5.36	50.3
REDUCTION FOR DEFICIT (IN)	0.00	0.00	-1.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CUM RED FOR DEFICIT (IN)	1.00	1.00	0.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
DEFICIT NOT SATISFIED (IN)	0.00	0.0	1.0	1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
REVISED IRRIG DEMAND (IN)	4.60	1.73	0.00	0.00	2.21	1.51	3.72	6.54	7.98	8.48	7.20	5.36	49.33
REVISED IRRIGATION DEMAND (MG)	168.1	63.1	0.0	0.0	80.9	55.1	136.2	239.2	291.8	310.0	263.3	196.1	1803.8
REVISED IRRIGATION DEMAND (MGD)	5.42	2.10	0.00	0.00	2.89	1.78	4.54	7.71	9.73	10.00	8.49	6.54	
LANDSCAPE IRRIGATION													
EVAPOTRANSPIRATION (IN)	3.22	1.64	0.65	0.85	2.25	2.28	3.52	4.73	5.66	5.93	5.06	3.75	39.6
IRRIG DEMAND = ET-PRECIP (IN)	3.22	1.03	0.00	0.00	2.25	0.53	2.21	4.51	5.55	5.93	5.03	3.75	34.0
REDUCTION FOR DEFICIT (IN)	0.00	0.00	-1.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CUM RED FOR DEFICIT (IN)	1.00	1.00	0.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
DEFICIT NOT SATISFIED (IN)	0.00	0.0	1.0	1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
REVISED IRRIG DEMAND (IN)	3.22	1.03	0.00	0.00	1.25	0.53	2.21	4.51	5.55	5.93	5.03	3.75	33.02
REVISED IRRIGATION DEMAND (MG)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
REVISED IRRIGATION DEMAND (MGD)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
INDUSTRIAL DEMAND (MG)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
EFFLUENT ROUTING ANALYSIS													
MAXIMUM POSSIBLE DISCHARGE TO CREEK (MG)	140.83	196.54	510.73	286.40	231.32	291.46	266.34	292.39	0.00	0.00	0.00	240.00	2455.99
TOTAL REUSE DEMAND (MG)	168.09	63.14	0.00	0.00	80.94	55.10	136.16	239.16	291.82	309.98	263.29	196.13	1803.79
MAXIMUM POSSIBLE DISCHARGE + REUSE (MG)	308.92	259.68	510.73	286.40	312.26	346.55	402.49	531.54	291.82	309.98	263.29	436.13	
VOLUME AVAILABLE FOR DISCHARGE + REUSE (MG)	244.72	243.78	343.43	289.33	235.63	287.71	286.54	256.81	235.99	238.41	239.48	233.52	
ACTUAL REUSE (MG)	168.09	63.14	0.00	0.00	80.94	55.10	136.16	239.16	235.99	238.41	239.48	196.13	1652.59
ACTUAL REUSE (MGD)	5.42	2.10	0.00	0.00	2.89	1.78	4.54	7.71	7.87	7.69	7.73	6.54	
REUSE DEMAND NOT SATISFIED (MG)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	55.83	71.56	23.81	0.00	151.20
REUSE DEMAND NOT SATISFIED (MGD)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.86	2.31	0.77	0.00	
ACTUAL DISCHARGE TO CREEK (MG)	76.63	180.64	343.43	286.40	154.69	232.61	150.38	17.65	0.00	0.00	0.00	37.39	1479.83
ACTUAL DISCHARGE TO CREEK (MGD)	2.47	6.02	11.08	9.24	5.52	7.50	5.01	0.57	0.00	0.00	0.00	1.25	
UNUSED DISCHARGE CAPACITY (MG)	64.20	15.90	167.30	0.00	76.63	58.84	115.96	274.73	0.00	0.00	0.00	202.61	976.17
UNUSED DISCHARGE CAPACITY (MGD)	2.07	0.53	5.40	0.00	2.74	1.90	3.87	8.86	0.00	0.00	0.00	6.75	
TSB OUTFLOW - TSB INFLOW (MG)	0.00	0.00	0.00	2.65	3.73	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
TERTIARY STORAGE BASINS													
BEGINNING STORAGE (MG)	0.00	0.00	0.00	5.59	3.73	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
BEGINNING WATER SURFACE AREA (AC)	35.40	35.40	35.40	35.58	35.52	35.40	35.40	35.40	35.40	35.40	35.40	35.40	
EVAP. VOLUME (MG)	0.00	0.75	0.96	0.71	0.00	2.15	1.61	0.27	0.13	0.00	0.04	0.00	6.61
PRECIP. VOLUME (MG)	0.00	0.75	6.55	1.51	0.00	2.15	1.61	0.27	0.13	0.00	0.04	0.00	13.00
STORAGE GAIN (MG)	0.00	0.00	5.59	-1.86	-3.73	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
FINAL STORAGE (MG)	0.00	0.00	5.59	3.73	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	

SUMMARY

ANNUAL INFLOW (MG)	
WASTEWATER WITHOUT I/I	2920
INFLOW AND INFILTRATION..	238
PRECIP. INTO PONDS/BASINS..	39
TOTAL..	3197

ANNUAL OUTFLOW (MG)	
DISCHARGE TO STREAM..	1480
TOTAL ALL REUSE...	1653
EVAP. FROM PONDS/BASINS..	64
TOTAL..	3197

INFLOW-OUTFLOW AND STORAGE (MG)	
ANNUAL INFLOW - ANNUAL OUTFLOW (MG)	**0**
STORAGE AVAILABLE (MG)..	190
STORAGE REQUIRED (MG)..	6
SURPLUS STORAGE CAPACITY (MG)..	**184**

CREEK DISCHARGE AND REUSE SUMMARY	
MAXIMUM POSSIBLE CREEK DISCHARGE (MG)	2456
ACTUAL CREEK DISCHARGE (MG)..	1480
UNUSED CREEK DISCHARGE CAPACITY (MG)..............................	**976**
REUSE DEMAND (MG)...	1804
ACTUAL REUSE (MG)..	1653
REUSE DEMAND NOT SATISFIED (MG)...	**151**

City of Lincoln WWTRF	WATER BALANCE - PROJECTED 8.0 MGD ADWF, WATER YR 2022 ALLOWABLE DISCHARGES BASED ON 15 MIN CALCS, WITH 2023 PERMIT REVISIONS WITH 1F SAFETY MARGIN OXIDATION DITCH TEMPERATURES USED	13-Apr-23 3:10 PM

OVERALL INPUT DATA

FLOWS AND INFILTRATION/INFLOWS (I/I)		CLIMATOLOGICAL AND RUNOFF FACTORS	
ADWF (MGD)...	8.00	OCT-APR EVAP/AVG EVAP RATIO..................	1.00
		MAY-SEP EVAP/AVG EVAP RATIO..................	1.00
		PAN COEFFICIENT..	0.80

PLANT SITE, MATURATION POND, AND TERTIARY STORAGE BASIN INPUT	MAT POND*	TERT STOR
RAIN CATCH AREA (AC) (*MAT POND + PLANT SITE).....................	95.0	46.2
MIN WATER SURFACE AREA (AC)..	40.0	35.4
MAX WATER SURFACE AREA (AC)..	40.0	41.4
MAX TERTIARY EFFL STORAGE (MG)..	---	190.0
LAND PRECIP COLLECTED (FRAC)...	0.90	0.90

IRRIGATION INPUT DATA	AGRICULTURE	LANDSCAPE
IRRIGATION AREA (AC)...	942.0	0.0
IRRIGATION EFFICIENCY (FRACTION)...	0.700	0.700
SOIL WATER DEFICIT BEFORE IRRIG. (IN)......................................	1.0	1.0

MONTHLY INPUT DATA

	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
DAYS IN MONTH	31	30	31	31	28	31	30	31	30	31	31	30	365
AVG PAN EVAP (IN)	4.89	2.06	1.25	0.92	1.90	3.47	5.21	8.07	9.91	11.12	9.93	7.45	66.18
WATER YEAR 2022 PRECIP (IN)	2.80	0.05	1.95	0.03	0.00	0.53	0.16	0.05	0.13	0.00	0.00	0.53	6.23
WATER YEAR 2022 Eto (IN)	3.53	1.51	0.72	1.80	2.92	4.22	5.43	7.53	8.20	8.31	7.51	5.56	57.24
AGRICULTURE CROP COEFF (ALFALFA)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
LANDSCAPE CROP COEFF (GRASS)	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	
INDUSTRIAL DEMAND (MGD)	0	0	0	0	0	0	0	0	0	0	0	0	
WYR 2022 MAX ALLOWABLE DISCH TO CREEK (MGD)	6.51	13.26	12.89	9.62	8.54	6.58	4.75	6.84	0.00	0.00	0.00	8.10	
WYR 2022 INFILTRATION AND INFLOW (I/I) (MGD)	2.27	1.01	4.64	1.61	0.54	0.60	1.21	0.45	0.31	0.00	0.00	0.03	

CALCULATIONS

	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	ANNUAL
INFLUENT INCLUDING I/I (MGD)	10.27	9.01	12.64	9.61	8.54	8.60	9.21	8.45	8.31	8.00	8.00	8.03	
EVAPORATION FROM PONDS (IN)	3.9	1.6	1.0	0.7	1.5	2.8	4.2	6.5	7.9	8.9	7.9	6.0	52.9
MATURATION POND													
INFLOW (MG)	318.28	270.29	391.96	298.00	239.17	266.62	276.43	261.82	249.36	248.00	248.00	240.96	3308.9
PRECIP. VOLUME (MG)	6.81	0.12	4.74	0.07	0.00	1.29	0.39	0.12	0.32	0.00	0.00	1.29	15.2
EVAP. VOLUME (MG)	4.25	1.79	1.09	0.80	1.65	3.02	4.53	7.02	8.62	9.67	8.63	6.48	57.5
OUTFLOW (MG)	320.84	268.62	395.61	297.27	237.52	264.89	272.28	254.92	241.06	238.33	239.37	235.77	3266.5
OUTFLOW (MGD)	10.35	8.95	12.76	9.59	8.48	8.54	9.08	8.22	8.04	7.69	7.72	7.86	
AGRICULTURE IRRIGATION													
EVAPOTRANSPIRATION (IN)	3.53	1.51	0.72	1.80	2.92	4.22	5.43	7.53	8.20	8.31	7.51	5.56	57.2
IRRIG DEMAND = ET-PRECIP (IN)	0.73	1.46	0.00	1.77	2.92	3.69	5.27	7.48	8.07	8.31	7.51	5.03	52.2
REDUCTION FOR DEFICIT (IN)	0.00	0.00	-1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CUM RED FOR DEFICIT (IN)	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
DEFICIT NOT SATISFIED (IN)	0.00	0.0	1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
REVISED IRRIG DEMAND (IN)	0.73	1.46	0.00	0.77	2.92	3.69	5.27	7.48	8.07	8.31	7.51	5.03	51.24
REVISED IRRIGATION DEMAND (MG)	26.8	53.4	0.0	28.0	106.8	135.1	192.7	273.7	295.1	303.9	274.5	183.8	1873.8
REVISED IRRIGATION DEMAND (MGD)	0.87	1.78	0.00	0.90	3.81	4.36	6.42	8.83	9.84	9.80	8.86	6.13	
LANDSCAPE IRRIGATION													
EVAPOTRANSPIRATION (IN)	2.47	1.06	0.50	1.26	2.04	2.96	3.80	5.27	5.74	5.82	5.25	3.89	40.1
IRRIG DEMAND = ET-PRECIP (IN)	0.00	1.01	0.00	1.23	2.04	2.43	3.64	5.22	5.61	5.82	5.25	3.36	35.6
REDUCTION FOR DEFICIT (IN)	0.00	0.00	-1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CUM RED FOR DEFICIT (IN)	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	
DEFICIT NOT SATISFIED (IN)	0.00	0.0	1.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
REVISED IRRIG DEMAND (IN)	0.00	1.01	0.00	0.23	2.04	2.43	3.64	5.22	5.61	5.82	5.25	3.36	34.61
REVISED IRRIGATION DEMAND (MG)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
REVISED IRRIGATION DEMAND (MGD)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
INDUSTRIAL DEMAND (MG)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
EFFLUENT ROUTING ANALYSIS													
MAXIMUM POSSIBLE DISCHARGE TO CREEK (MG)	201.73	397.89	399.64	298.12	239.17	203.85	142.40	212.00	0.00	0.00	0.00	243.04	2337.84
TOTAL REUSE DEMAND (MG)	26.82	53.39	0.00	28.04	106.78	135.06	192.72	273.65	295.11	303.88	274.51	183.82	1873.76
MAXIMUM POSSIBLE DISCHARGE + REUSE (MG)	228.55	451.28	399.64	326.15	345.95	338.91	335.12	485.65	295.11	303.88	274.51	426.86	
VOLUME AVAILABLE FOR DISCHARGE + REUSE (MG)	320.84	360.58	395.61	298.70	237.52	264.89	272.28	254.92	241.06	238.33	239.37	235.77	
ACTUAL REUSE (MG)	26.82	53.39	0.00	28.04	106.78	135.06	192.72	254.92	241.06	238.33	239.37	183.82	1700.29
ACTUAL REUSE (MGD)	0.87	1.78	0.00	0.90	3.81	4.36	6.42	8.22	8.04	7.69	7.72	6.13	
REUSE DEMAND NOT SATISFIED (MG)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	18.73	54.05	65.55	35.14	0.00	173.47
REUSE DEMAND NOT SATISFIED (MGD)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.60	1.80	2.11	1.13	0.00	
ACTUAL DISCHARGE TO CREEK (MG)	201.73	307.19	395.61	270.67	130.74	129.83	79.57	0.00	0.00	0.00	0.00	51.95	1567.29
ACTUAL DISCHARGE TO CREEK (MGD)	6.51	10.24	12.76	8.73	4.67	4.19	2.65	0.00	0.00	0.00	0.00	1.73	
UNUSED DISCHARGE CAPACITY (MG)	0.00	90.70	4.02	27.45	108.43	74.01	62.84	212.00	0.00	0.00	0.00	191.09	770.55
UNUSED DISCHARGE CAPACITY (MGD)	0.00	3.02	0.13	0.89	3.87	2.39	2.09	6.84	0.00	0.00	0.00	6.37	
TSB OUTFLOW - TSB INFLOW (MG)	-92.29	91.96	0.00	1.43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
TERTIARY STORAGE BASINS													
BEGINNING STORAGE (MG)	0.00	91.96	0.00	1.43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
BEGINNING WATER SURFACE AREA (AC)	35.40	38.30	35.40	35.45	35.40	35.40	35.40	35.40	35.40	35.40	35.40	35.40	
EVAP. VOLUME (MG)	3.76	0.06	0.96	0.04	0.00	0.65	0.20	0.06	0.16	0.00	0.00	0.65	6.54
PRECIP. VOLUME (MG)	3.43	0.06	2.39	0.04	0.00	0.65	0.20	0.06	0.16	0.00	0.00	0.65	7.64
STORAGE GAIN (MG)	91.96	-91.96	1.43	-1.43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
FINAL STORAGE (MG)	91.96	0.00	1.43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	

SUMMARY

ANNUAL INFLOW (MG)	
WASTEWATER WITHOUT I/I	2920
INFLOW AND INFILTRATION..	389
PRECIP. INTO PONDS/BASINS..	23
TOTAL..	3332

ANNUAL OUTFLOW (MG)	
DISCHARGE TO STREAM...	1567
TOTAL ALL REUSE..	1700
EVAP. FROM PONDS/BASINS..	64
TOTAL..	**3332**

INFLOW-OUTFLOW AND STORAGE (MG)	
ANNUAL INFLOW - ANNUAL OUTFLOW (MG)	**0**
STORAGE AVAILABLE (MG)...	190
STORAGE REQUIRED (MG)..	92
SURPLUS STORAGE CAPACITY (MG)...	**98**

CREEK DISCHARGE AND REUSE SUMMARY	
MAXIMUM POSSIBLE CREEK DISCHARGE (MG)	2338
ACTUAL CREEK DISCHARGE (MG)...	1567
UNUSED CREEK DISCHARGE CAPACITY (MG)..............................	**771**
REUSE DEMAND (MG)..	1874
ACTUAL REUSE (MG)...	1700
REUSE DEMAND NOT SATISFIED (MG)..	**173**

APPENDIX B

LiSWA WWTRF Phase 1 Improvement Project – Maturation Pond Effluent Pump Station Design Report, by Stantec, May 2024

LiSWA WWTRF Phase 1 Improvement Project – Maturation Pond Effluent Pump Station

Design Report

Prepared for:
City of Lincoln

Prepared by:
Stantec Consulting Services Inc.

May 31, 2024

Sign-off Sheet

This document entitled LiSWA WWTRF Phase 1 Improvement Project – Maturation Pond Effluent Pump Station was prepared by Stantec Consulting Services Inc. ("Stantec") for the City of Lincoln (the "Client"). Any reliance on this document by any third party is strictly prohibited. The material in it reflects Stantec's professional judgment in light of the scope, schedule and other limitations stated in the document and in the contract between Stantec and the Client. The opinions in the document are based on conditions and information existing at the time the document was published and do not take into account any subsequent changes. In preparing the document, Stantec did not verify information supplied to it by others. Any use which a third party makes of this document is the responsibility of such third party. Such third party agrees that Stantec shall not be responsible for costs or damages of any kind, if any, suffered by it or any other third party as a result of decisions made or actions taken based on this document.

Prepared by ______________________________
(signature)

Breanna Webb, EIT

Reviewed by ______________________________
(signature)

Gabe Aronow, PE

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1.0 PURPOSE AND SCOPE

The purpose of this Design Report is to describe the new Maturation Pond Effluent Pump Station (MPEPS) at the Lincoln-SMD1 Wastewater Authority (LiSWA) Wastewater Treatment and Reclamation Facility (WWTRF). The pump station is needed to utilize additional storage volume in the maturation ponds, as described in the report titled *Lincoln WWTRF Review of Maturation Pond and Tertiary Storage Operation and Sizing and Impacts on Other Facilities Based on Updated Data and New Permit Temperature Requirement* (April 2023). The 2023 report documents the need for additional available storage volume (and associated requirements for maturation pond water level lowering) and increasing the discharge rate to the tertiary treatment facilities.

This report presents the design concepts for the new MPEPS and is divided into the following sections:

- Existing Facilities
- Updated Design Criteria
- Pump Alternatives
- Preliminary Design
- Conclusions & Recommendations

2.0 EXISTING FACILITIES

The existing maturation ponds are used to normalize priority pollutant concentrations before processing through downstream treatment facilities. They also equalize influent peak flows, allowing a reduced flow rate to be conveyed to downstream facilities. Effluent from the maturation ponds discharge through two existing maturation pond outlet structures before reaching the maturation pond level control structure, where it is diverted to the Dissolved Air Floatation (DAF) tanks for further treatment.

Currently, flow from the maturation ponds is primarily conveyed by gravity through the maturation pond outlet structures. When levels in the ponds are too low for gravity flow, two existing submersible pumps (25 HP, Xylem/Flygt NP3171-614LT) within the outlet structures are used to convey additional flow. However, these pumps were originally included as maturation pond drain pumps and have limited capacity. The new MPEPS will expand this pumping capacity to accommodate the overall WWTRF Improvements Project, covering the Phase 1, Phase 2 and Phase 3 expansion planning requirements, increasing the effluent flow rate and achievable low water level in the maturation ponds.

3.0 UPDATED DESIGN CRITERIA

The new MPEPS and existing maturation pond outlet structure pumps needs to convey a total of 19.32 MGD from a low water elevation of 101.3 feet and a minimum flow of approximately 1.0 MGD. The MPEPS design criteria are shown in **Table 1**.

Table 1 Maturation Ponds Effluent Pump Station Design Criteria

Parameter	Updated Criteria
Total Flow, Combined Pumping Capacity (MGD) (1)	19.32
Total Pumping Capacity (gpm) (1)	13,417
Low Water Level, LWL (ft) (2)	101.3
Maximum Surface Level, MSL (ft) (2)	114.0
Total Dynamic Head, TDH (ft)	13.3

1. Total required pumping capacity including the existing maturation pond outlet structure pumps.
2. Water levels required in the MPEPS.

4.0 PUMP ALTERNATIVES

Stantec considered the following pumps and design alternatives for the Maturation Pond Effluent Pump Station:

- Alternative 1: Flygt Axial Flow Propeller Pumps (PL7030)
- Alternative 2: Flowserve Axial Flow Vertical Pumps (15AFV-DL)
- Alternative 3: Flygt Submersible Pumps (NP3171) to Match Existing Outlet Structure Pumps

Alternative 1 – Flygt: PL7030

The Flygt submersible vertically installed axial flow pumps are installed in a vertical discharge tube on a support flange. This alternative did not meet the minimum flow requirements for the lift station. These large pumps could not be turned down to reach the minimum flow requirement of 1.0 MGD.

Alternative 2 – FlowServe: 15AFV-DL

The Flowserve AFV axial flow suspended shaft vertical pump is a single stage propeller type design. This alternative was dismissed because the discharge header could not be located below deck, allowing the potential for gravity flow through the pump (with pumps off), and the elevated discharge header would incur additional head loss. The pump station structure would also be larger, incurring added construction costs.

Alternative 3 – Flygt: NP3171

Each of the maturation pond outlet structures house a single Flygt NP3171 pump, these pumps are efficient and meet the head range requirements effectively. Three more of these pumps are needed to meet the design criteria required for the new lift station, combined. After considering many pumps and manufacturers, more of the existing Flygt NP3171, in conjunction with the existing pumps, appears to be best MPEPS option.

5.0 PRELIMINARY DESIGN

The following sections describe the recommended MPEPS design.

Pumps

The recommended design includes installing three new 25 HP Flygt NP3171 pumps in a new MPEPS wet well structure, with a slot for a future fourth pump, in addition to the continued operation of the two existing maturation pond outlet structure pumps. Based on discussions with WWTRF operators the existing pumps have a maximum pumping capacity of approximately 8.0 MGD (4.0 MGD each). The new pumps will have a pumping capacity of approximately 5.1 MGD (3550 gpm) each. This is slightly higher than the existing pumps due to the losses associated with the discharge piping from the outlet structures.

The new station will have a reliable pumping capacity of approximately 10.22 MGD and a maximum pumping capacity of approximately 15.34 MGD. The combined reliable capacity of the new station and the existing pumps meets the capacity requirements of the MPEPS of 19.32 MGD total. If one of the existing pumps is considered the redundant pump, the combined reliable capacity falls short of the design requirement at 18.22 MGD. Therefore, during the Phase 2 or Phase 3 expansion projects another pump should be added to the fourth slot in the MPEPS to ensure the combined reliable capacity meets the total pumping requirements under future conditions.

Flygt recommends that the pumps not pump less than 1.0 MGD and that the maximum flow be capped at approximately 5.1 MGD. The system and pump curves for the Flygt NP3171 pump at various speeds within the MPEPS are shown in **Figure 1.**

The pump parameters are summarized in **Table 2** and the cut sheet for the NP3171 pump is included in this report as **Appendix A**.

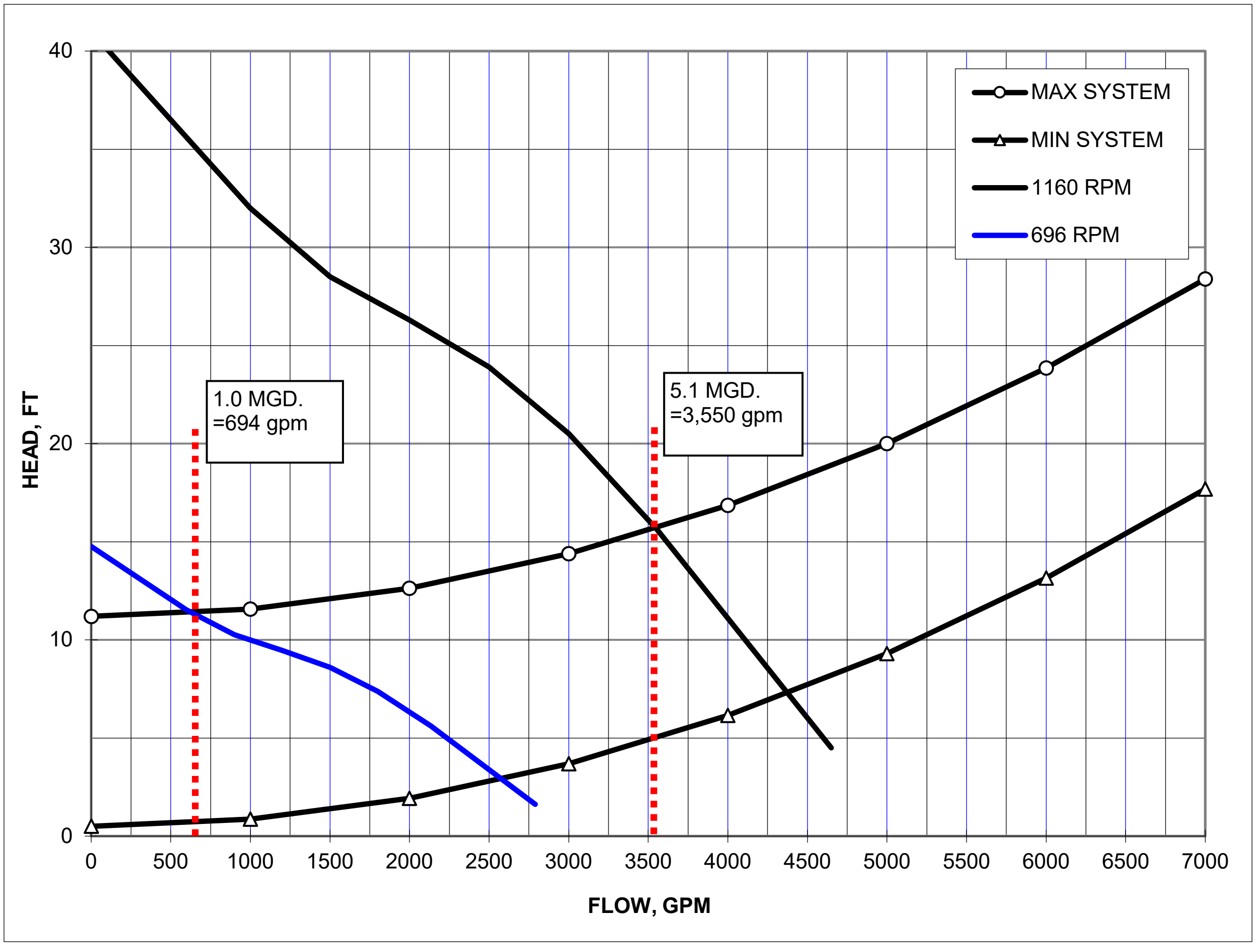


Figure 1 MPEPS Pump and System Curves

Table 2 Pump Station Design Parameters

	Pump Station
Number of Units	4 Duty, 1 Standby (2 are existing)
Operating Characteristics,	
Flow, gpm/TDH, ft. of water (design point)	3550/16 [(a)]
Discharge size, inches	10
Motor size, Hp	25
Maximum speed, rpm	1,160
Minimum Bowl efficiency at design point, %	78

(a) Existing pumps have slightly reduced capacity due to the discharge piping.

Station Layout

The pump station design will accommodate higher flow rates that may occur under future conditions with the installation of a fourth pump and/or larger pumps. The new pumps spacing dictates the overall width of the station to be 20-feet, to allow for a spacing of 5-feet between pump centerlines, based on Hydraulic Institute standards (ANSI/HI 9.8). The bottom of the lift station will be approximately 21.7-feet deep, to provide the required operating depth in the maturation pond and maintain minimum submergence conditions required for the pumps. The internal baffle wall openings will be 30-inches by 20-inches to ensure pump approach velocities are sufficiently low at peak flow rates.

Piping

New inlet piping into the MPEPS from the existing maturation ponds are incorporated into the design concept. The new piping will avoid creating undesirable flow vortices near the existing pumps that would otherwise occur by connecting the new wetwell to the existing outlet structures. Two new 36-inch lines from the maturation ponds will tee together into a 42-inch line into the MPEPS. The MPEPS outlet pipe connecting to the level control structure will need to be 48-inches, to accommodate the potential for higher flows under future conditions. A hydraulic control gate will be installed on this pipe to isolate downstream infrastructure, similar to the existing outlet control structure.

Flow Meter

The new flow meter installed on the existing 48-inch Maturation Pond effluent pipe (from the level control structure to the DAFs) will be 36-inches in diameter and capable of accommodating the full flow range of flows (1MGD to 19.3 MGD) from the maturation ponds to downstream facilities. Due to the limited space for a straight pipe run before and after the meter an ABB MagMaster MFE or Toshiba "Mount Anywhere" flow meter will be used in the design. These meters maintain a high level of accuracy with limited hydraulic conditions.

Plan and section views of the proposed MPEPS and associated structures are shown in **Figure 2** and **Figure 3**.

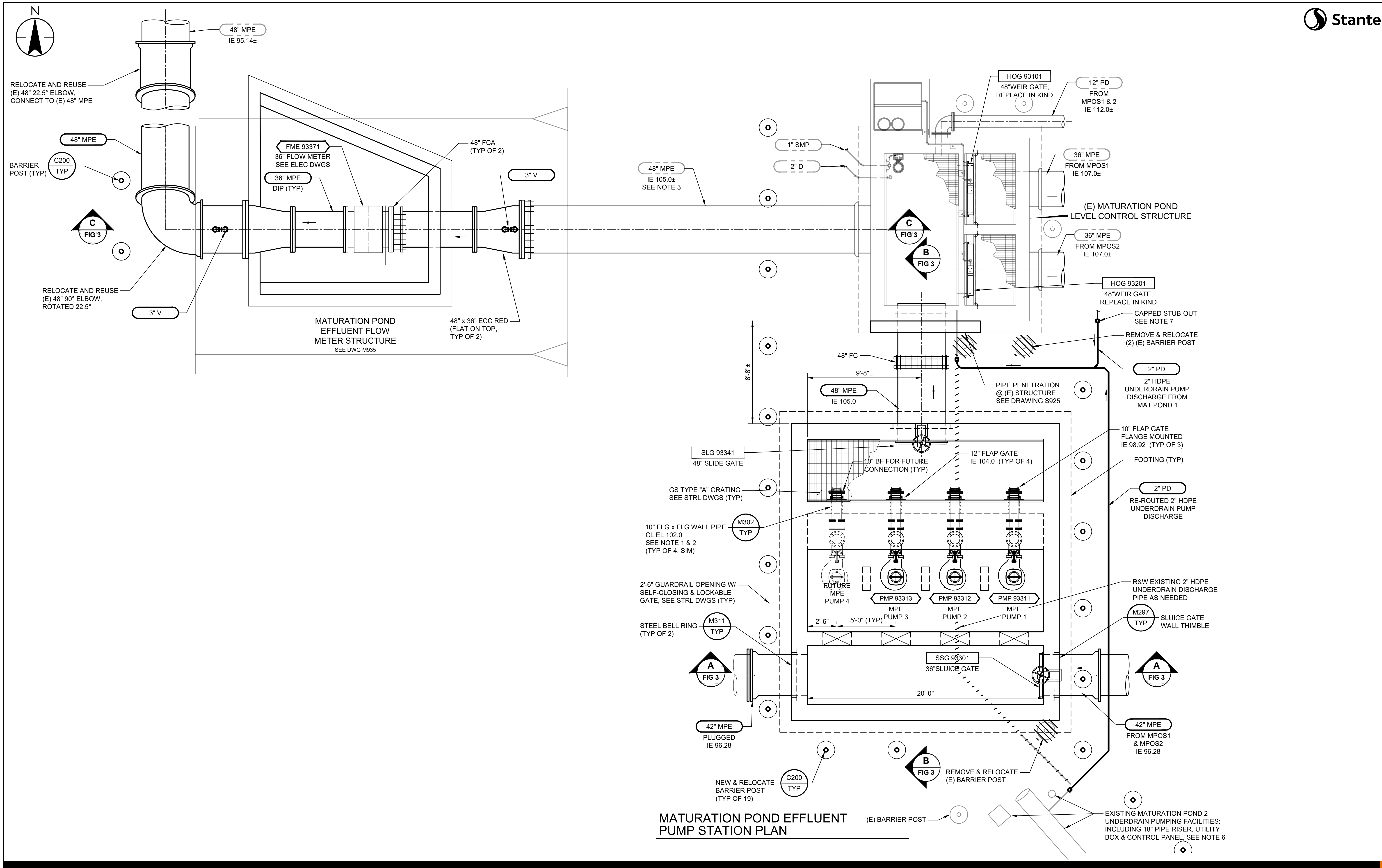


Figure 2
Maturation Pond Effluent Pump Station and Flow Meter Vault Plan

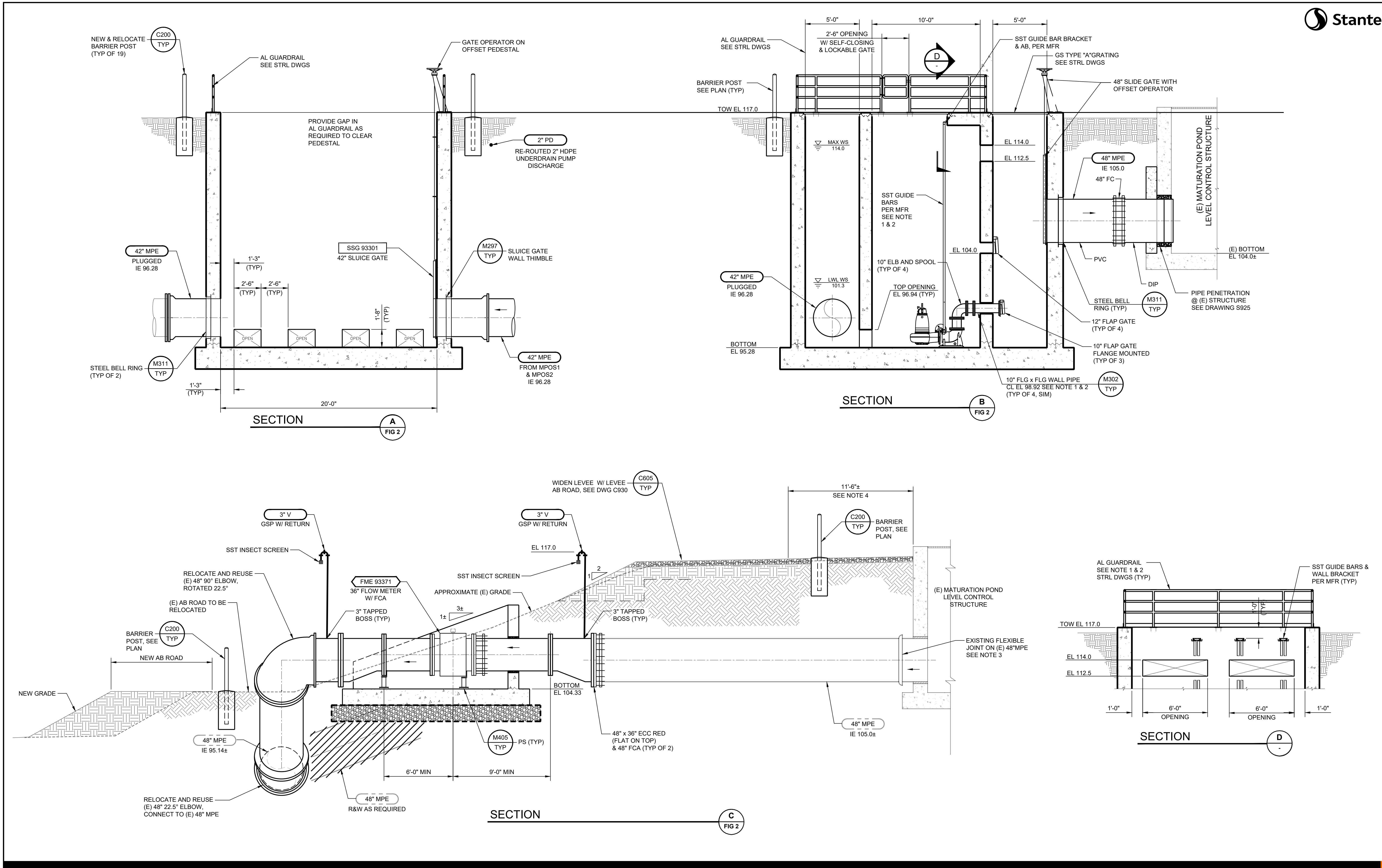


Figure 3

Maturation Pond Effluent Pump Station and Flow Meter Vault Sections

6.0 CONCULSIONS & RECOMMENDATIONS

The best apparent design of the MPEPS includes continued use of the outlet structure pumps with installation of like pumps in the new MPEPS. The design will require two new 36-inch inlet pipes with new pipe penetrations into the maturation ponds that tee together into a 42-inch inlet pipe into the MPEPS. The outlet pipe from the new MPEPS to the existing control structure will be 48-inch. The pump station will have room for four Flygt NP3171 pumps. Three pumps will be installed with the Phase 1 project to provide a reliable capacity of approximately 19.32 MGD, including the capacity of the existing pumps.

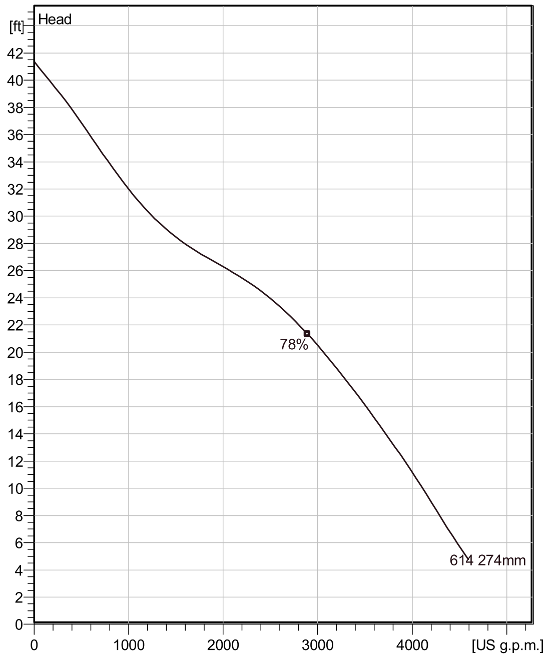
Appendix A FLYGT NP3171 CUT SHEET

NP 3171 LT 3~ 614

Patented self cleaning semi-open channel impeller, ideal for pumping in waste water applications. Modular based design with high adaptation grade.

Technical specification

Curves according to: Water, pure Water, pure [100%],39.2 °F,62.42 lb/ft³,1.6891E-5 ft²/s



Curve: ISO 9906

Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

Configuration

Motor number
N3171.095 25-18-6BB-W
25hp

Installation type
P - Semi permanent, Wet

Impeller diameter
274 mm

Discharge diameter
10 inch

Pump information

Impeller diameter
274 mm

Discharge diameter
10 inch

Inlet diameter
250 mm

Maximum operating speed
1160 rpm

Number of blades
2

Max. fluid temperature
40 °C

Material

Impeller
Hard-Iron ™

Project	Xylect-22017590	**Created by**	David Troyer		
Block		**Created on**	3/18/2024	**Last update**	3/18/2024

NP 3171 LT 3~ 614

Technical specification

Motor - General

Motor number N3171.095 25-18-6BB-W 25hp	**Phases** 3~	**Rated speed** 1160 rpm	**Rated power** 25 hp
ATEX approved FM	**Number of poles** 6	**Rated current** 32 A	**Stator variant** 7
Frequency 60 Hz	**Rated voltage** 460 V	**Insulation class** H	**Type of Duty** S1
Version code 095			

Motor - Technical

Power factor - 1/1 Load 0.86	**Motor efficiency - 1/1 Load** 86.4 %	**Total moment of inertia** 6 lb ft²	**Starts per hour max.** 30
Power factor - 3/4 Load 0.81	**Motor efficiency - 3/4 Load** 88.0 %	**Starting current, direct starting** 173 A	
Power factor - 1/2 Load 0.71	**Motor efficiency - 1/2 Load** 88.1 %	**Starting current, star-delta** 57.7 A	

Project	Xylect-22017590	**Created by**	David Troyer		
Block		**Created on**	3/18/2024	**Last update**	3/18/2024

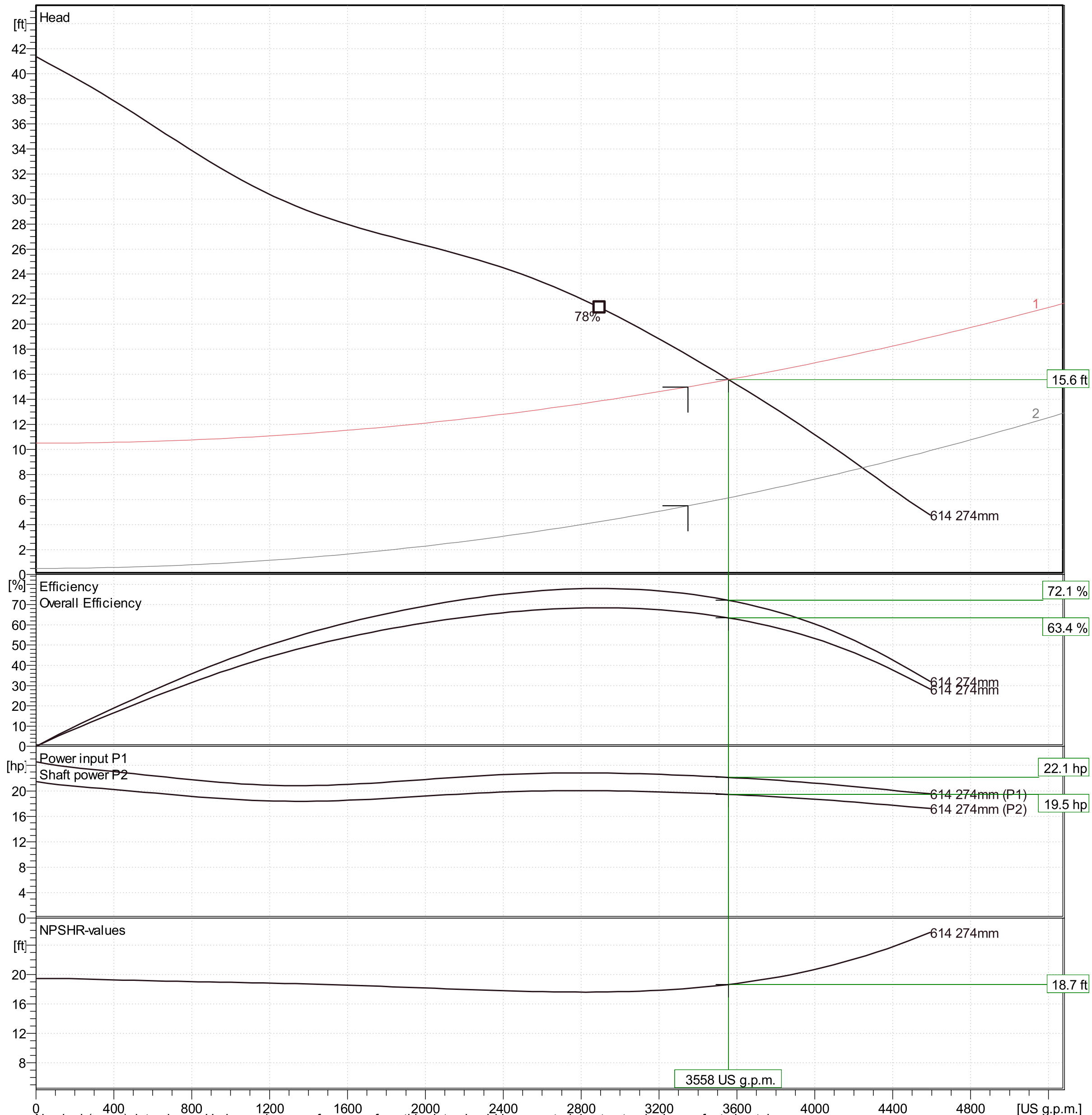
NP 3171 LT 3~ 614

Performance curve

Duty point

Flow	Head
3560 US g.p.m.	15.6 ft

Curves according to: Water, pure Water, pure [100%],39.2 °F,62.42 lb/ft³,1.6891E-5 ft²/s



Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances. Please consult your local Flygt representative for performance guarantees.

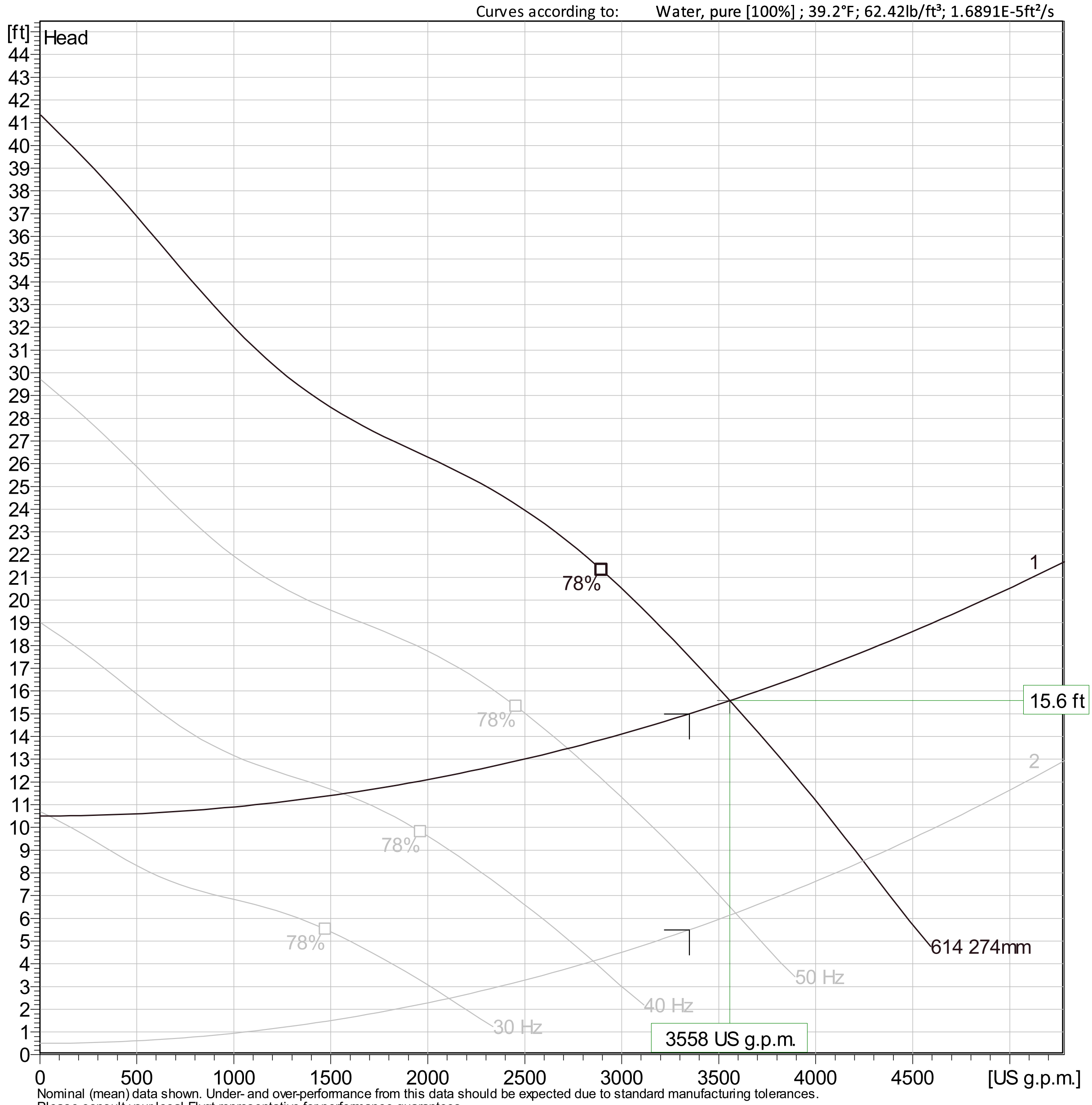
Curve: ISO 9906

Xylect-22017590		David Troyer		
	Created on	3/18/2024	**Last update**	3/18/2024

NP 3171 LT 3~ 614

VFD Analysis

Curves according to: Water, pure [100%] ; 39.2°F; 62.42lb/ft³; 1.6891E-5ft²/s



Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

Operating Characteristics

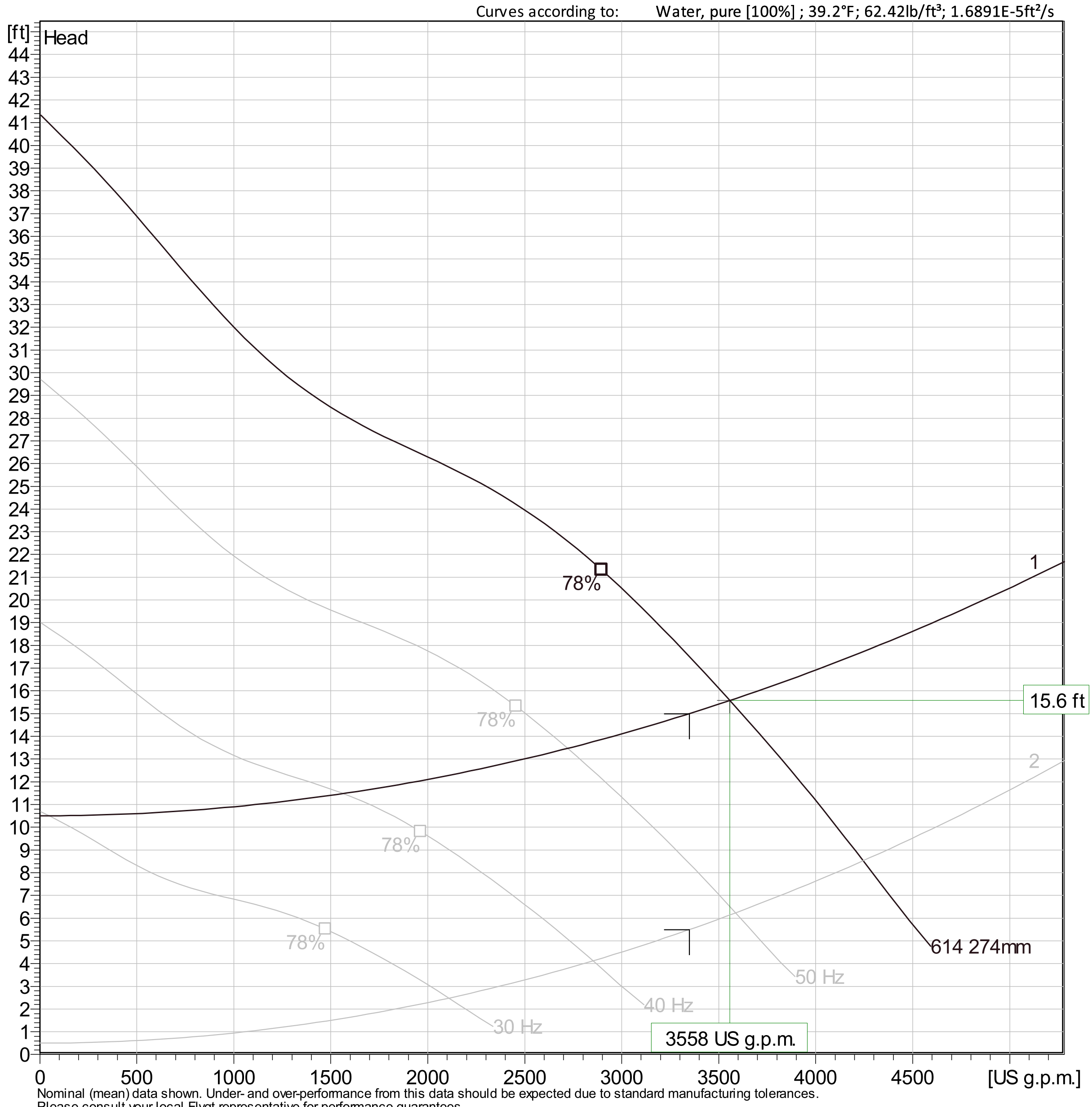
Pumps / Systems	Frequency	Flow	Head	Shaft power	Flow	Head	Shaft power	Hydr.eff.	Specific energy	NPSHre
		US g.p.m.	ft	hp	US g.p.m.	ft	hp		kWh/US MG	ft
2	59 Hz	4240	8.53	18.1	4240	8.53	18.1	50.4 %	60.2	22.4
2	50 Hz	3590	6.23	11.1	3590	6.23	11.1	51 %		17.1
2	40 Hz	2850	4.12	5.69	2850	4.12	5.69	52.2 %		11.9
2	30 Hz	2110	2.48	2.42	2110	2.48	2.42	54.7 %		7.35

Project	Xylect-22017590	**Created by**	David Troyer		
Block		**Created on**	3/18/2024	**Last update**	3/18/2024

NP 3171 LT 3~ 614

VFD Analysis

Curves according to: Water, pure [100%] ; 39.2°F; 62.42lb/ft³; 1.6891E-5ft²/s



Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

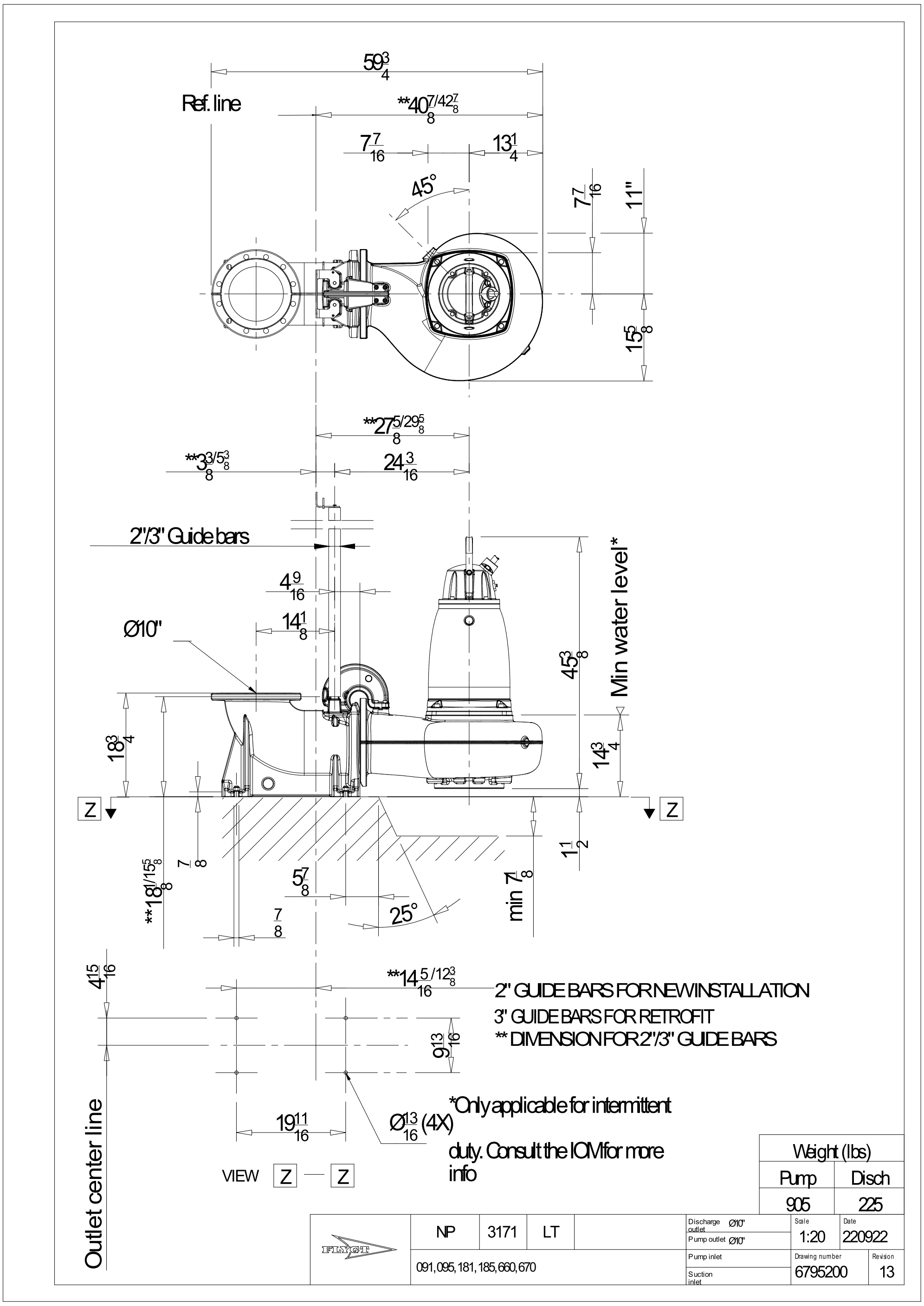
Operating Characteristics

Pumps / Systems	Frequency	Flow	Head	Shaft power	Flow	Head	Shaft power	Hydr.eff.	Specific energy	NPSHre
		US g.p.m.	ft	hp	US g.p.m.	ft	hp		kWh/US MG	ft
1	59 Hz	3560	15.6	19.5	3560	15.6	19.5	72.1 %	77.3	18.7
1	50 Hz	2720	13.5	12.1	2720	13.5	12.1	76.7 %		13.7
1	40 Hz	1560	11.5	6.15	1560	11.5	6.15	74 %		9.6
1	30 Hz	47.4	10.5	2.77	47.4	10.5	2.77	4.71 %		6.6

Project	Xylect-22017590	**Created by**	David Troyer		
Block		**Created on**	3/18/2024	**Last update**	3/18/2024

NP 3171 LT 3~ 614

Dimensional drawing



Project	Xylect-22017590	**Created by**	David Troyer
Block		**Created on**	3/18/2024 **Last update** 3/18/2024

APPENDIX C

LiSWA UV Basis of Design, by Stantec, August 2024

LiSWA UV Basis of Design
Prepared by: Kelly Valencia, EIT
Reviewed by: Cristina Fonseca, PE
Electrical & Instrumentation Review by: Javier Fernandez, PE
Date: 8/16/2024

1 Ultraviolet (UV) Disinfection System Design

1.1 Existing UV Disinfection System

The ultraviolet (UV) disinfection system currently installed at the Lincoln-SMD1 Wastewater Authority (LiSWA) Wastewater Treatment and Reclamation Facility (WWTRF) is comprised of five open channels each equipped with Wedeco (a Xylem brand) TAK55 UV disinfection equipment, complete with an in-channel cleaning system and control equipment. Each channel has five banks (four duty plus one standby bank) with low-pressure, high-intensity lamps. The existing UV disinfection system is capable of delivering a dose of 100 mJ/cm^2 at a design flow of 17.5 Mgal/day and a design minimum ultraviolet transmittance (UVT) of 70%. An additional sixth channel, currently sitting empty, was built to accommodate future flows. The UV disinfection system provides final disinfection of the tertiary-filtered effluent prior to disposal and/or reuse.

1.2 UV Disinfection System Expansion

The UV disinfection system is planned for expansion as part of the LiSWA WWTRF Phase 1 Improvement Project. The UV disinfection system will be expanded in kind with the newest version of the Wedeco TAK55 system. All six UV channels will receive new UV equipment (i.e., banks, modules, lamps, quartz sleeves, ballasts, pneumatically driven automatic wiping system, etc.). The UV disinfection system is designed to deliver a minimum UV dose of 100 mJ/cm^2 (matching the current design and the permitted minimum hourly average UV dose) at a design minimum UVT of 70% (matching the current design). The design capacity of the system is based on six duty channels each with four duty banks and one standby bank, which is one of the two types of redundancy recommended by the Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse (National Water Research Institute [NWRI] in collaboration with Water Research Foundation, August 2012, Third Edition), hereafter referred to as the 2012 UV Guidelines. Each bank will have 3 modules with 12 lamps per modules, which equates to 180 lamps per channel (144 duty plus 36 standby) and 1,080 lamps total (864 duty plus 216 standby). The expansion project will increase the capacity of the UV disinfection system to meet the peak month flow conditions plus in-plant recycle flows (20.6 Mgal/d total).

The UV disinfection system design criteria for the expansion are summarized in **Table 1**.

Table 1 UV Disinfection System Expansion Design Criteria

Design Criteria	Value
Manufacturer / Model	Wedeco / TAK55 H (110 mm lamp centerline spacing) (1)
Peak Month Flow + In-Plant Recycle Flows	20.6 Mgal/d
UV Disinfection System Design Peak Flow Capacity	3.6 Mgal/d per channel (21.6 Mgal/d total) (1)(2)
Design Minimum UV Dose	100 mJ/cm^2
Design Minimum UV Transmittance (UVT)	70% @ 254 nm
Channels	6 (6 duty)
Banks per Channel	5 (4 duty, 1 standby)
Modules per Bank	3
Lamps per Module	12
Lamps per Channel	180 (144 duty, 36 standby)
Total Number of Lamps in System	1,080 (864 duty, 216 standby)
Design End of Lamp Life (EOLL) Value	0.87 (guaranteed lamp life of 14,000 hours) (2)
Design Fouling Factor (FF) Value	0.80 (3)
Effluent Finger Weir Length / Top Elevation	720 inches (60 feet, total perimeter) / 107.81 feet (4)
Required Channel Width	25 13/16 inches (5)
Effluent Total Coliform Permit Requirements	<2.2 MPN/100 mL (7-day median) <23 MPN/100 mL (cannot exceed more than once in any 30-day period) <240 MPN/100 mL (at all times)

1. See TAK55 validation details in Section 1.2.1.
2. Ecoray ELR-30 lamps have a third party validated end of lamp life (EOLL) of 0.87 for 14,000 hours of operation. Stantec has contacted the Division of Drinking Water (DDW) to request approval to use a design EOLL of 0.87. The peak flow capacity presented in this table assumes that DDW will approve using a design EOLL of 0.87. See Section 1.2.1.1 for further detail.
3. The current design capacity is based on a fouling factor (FF) of 0.80. DDW indicated that an onsite fouling study would be needed to increase the design FF. See Section 1.2.1.2 for further detail.
4. The effluent finger weirs are required to be replaced to increase the weir length and lower the top of weir elevation. Wedeco provided a preliminary total weir length and top of weir elevation. The final values shall be confirmed by Wedeco.
5. The TAK55 system with the 110 mm lamp centerline spacing has a required channel width of 25 13/16 inches. The width of the existing channels (currently 28 inches) will be reduced using 304 stainless steel plates on both sides of the channel (to protect the coating on the channel walls). Refer to drawings for additional information.

1.2.1 VALIDATION IMPROVEMENTS

The existing LiSWA UV disinfection system original design and associated system capacity was based on the validation report (WEDECO Ultraviolet Technologies TAK-55HP VALIDATION REPORT, FINAL; Carollo Engineers, October 2003), which summarized the performance validation testing of a pilot scale system operated at the City of Roseville Dry Creek Wastewater Reclamation Plant. This validation report meets the requirements of the Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse (National Water Research Institute and American Water Works Association Research Foundation [NWRI/AWWARF], May 2003, Second Edition). The new TAK55 system planned to replace the current system as part of the WWTRF expansion is based on the validation report (Wedeco Open Channel TAK-55

Wastewater UV Reactor 320W Validation Report; Carollo Engineers, January 2010), which meets the requirements of the most recent 2012 UV Guidelines.

In addition to improvements in technology, the new design has a UV lamp centerline distance of 110 mm compared to the 120 mm UV lamp centerline spacing of the older model. This allows for improved overall UV disinfection system performance.

1.2.1.1 End of Lamp Life

The UV disinfection system currently in operation at the LiSWA WWTRF calculates dose delivery using an end of lamp life (EOLL) factor 0.85. The current design, which uses low-pressure high-output (LPHO) Ecoray ELR-30 lamps, assumes a less conservative EOLL factor of 0.87. This value has been selected based on the following considerations:

- Ecoray ELR-30 lamps have third party validated EOLL values of 0.90 for 12,048 hours of operation (report by Dr.-Ing M. Groebel, July 2011) and 0.87 for 14,000 hours of operation (report by Dr.-Ing M. Groebel, March 2012).
- The Division of Drinking Water (DDW) preliminarily approved use of the EOLL of 0.90 for 12,000 hours of use.

Stantec has contacted DDW to confirm that using a design EOLL of 0.87 for 14,000 hours of operation is also approved to increase the hours of operation allowed before the lamps are required to be replaced. Since DDW gave preliminary approval to use the higher EOLL of 0.90, it is likely that DDW will approve using a design EOLL of 0.87. Therefore, the EOLL of 0.87 was assumed for the current WWTRF UV expansion design.

The use of the lower EOLL factor, although limiting the design flow, benefits the WWTRF in terms of life-cycle costs. In the future, as the peak flows increase, the higher EOLL factor (with reduced lamp hours of operation) can be considered.

If DDW does not approve using a design EOLL of 0.87, then a design EOLL of 0.90 can be used, which would slightly increase the design capacity and decrease the hours of operation before the lamps must be replaced.

1.2.1.2 Fouling Factor

The system currently in place at the LiSWA was sized based on a fouling factor (FF) of 0.80. A sleeve fouling test was conducted to assess the performance of the Wedeco mechanical wiping system (analysis review presented in the report, Sleeve Fouling Study Summary Report, November 2009). As a result of the this, Carollo Engineers provided a Sleeve Fouling Certificate dated April 12, 2013 that states that a FF of 0.958 was determined for the Wedeco mechanical wiping system. However, DDW indicated that the default FF is 0.80, and an onsite fouling study would be needed to increase the design FF. If onsite studies are carried to substantiate a higher FF, this value can be revisited in the future.

1.2.2 INSTRUMENTATION & PLC REDUNDANCY

The existing UV system is currently controlled by two Programmable Logic Controllers (PLCs) that provide automation for six UV channels. PLC-401 controls Channels 1-3 as duty channels and PLC-402 controls Channels 4 and 5 (with future Channel 6) as standby when additional disinfection or capacity is required. Although there are two PLC units controlling separate channels, PLC-402 is dependent on PLC-401 for analytical data required to operate Channel 4 and 5. The current configuration does not permit the two PLCs to independently control each set of channels and depend on a single PLC and point-of-failure.

A new control scheme and strategy is proposed and coordinated with LiSWA WWTRF operations team as part of the facility upgrade.

The following equipment will be replaced to improve redundancy and increase operational flexibility:

- PLCs and enclosures;
- UV equipment including control cabinets, ballasts, ballast enclosures, ballast distribution, lamp-to-ballast cables, and junction boxes; and
- Instrumentation, including the high/low water level sensor, ultrasonic water level sensor, and UVT meter (the YSI meter will be replaced with a Hach meter).

The existing UV system container that currently houses the control panels and electrical equipment will remain. The air conditioning units currently installed were determined to be sufficient for the new system loads and will remain.

A new control cabinet, also referred to as Instrumentation Control Automation (ICA)-600 UL, with fully redundant Allen Bradley ControlLogix PLC will be provided to operate channel configuration independently and to improve reliability and flexibility. The PLC improvements will also allow Channels 4 through 6 to be operated independently of Channels 1 through 3. The redundant PLC will provide continuous control of the UV system should the master PLC fail. The ICA enclosure will be equipped with an uninterruptible power supply (UPS) to provide up-to 15 minutes of back-up power.

The PLC will also include a communication module to import and export all UV data from/to the Supervisory Control and Data Acquisition (SCADA) system via Ethernet/IP. Ethernet/Ip capability will mainstream data flow to the SCADA and the servers.

APPENDIX D.1

Geotechnical Design Report Update, Lincoln WWTRF Phase 1 and Phase 2 Expansion Project, WWTP Improvements, by Blackburn Consulting, June 2024

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Auburn, CA 95603
(530) 887-1494

BLACKBURN CONSULTING

Fresno (530) 887-1494
West Sacramento (916) 375-8706

File No. 3228.X
June 4, 2024

Mr. Gabe Aronow, P.E.
Stantec
3875 Atherton Road
Rocklin CA 95765

Subject: **GEOTECHNICAL DESIGN REPORT UPDATE, WWTRF IMPROVEMENTS, REV 1**
Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
Placer County, California

Dear Mr. Aronow:

Blackburn Consulting is pleased to submit this Geotechnical Report Update letter for the proposed Lincoln Wastewater Treatment and Reclamation Facility (LWWTRF) Phase 1 and Phase 2 Expansion Project located in Placer County, California.

This addendum updates our April 10, 2018, Geotechnical Design Report recommendations for the wastewater treatment plant expansion. We understand the proposed type and location of improvements has not changed since our original report. We still consider our previous report recommendations appropriate unless specifically modified in this addendum.

SCOPE

To prepare this addendum, Blackburn reviewed our April 10, 2018, Geotechnical Design Report for the LWWTRF Phase 1 and 2 Expansion Project and updated the seismic design parameters.

UPDATED RECOMMENDATIONS

2022 California Building Code Seismic Parameters

Blackburn used the following to update the seismic (CBC) design parameters:

- SEAOC/OSHPD Seismic Design Maps Tool
- ASCE 7-16 Reference Standard
- Risk Category 2
- Site Class C – Very Dense Soil
- Latitude: 38.863059 Longitude: -121.346659

We selected these inputs based on the subsurface conditions in the borings and measured blow counts. Table 4 presents our updated 2022 CBC seismic design parameters.

Table 1: 2022 CBC Seismic Design Parameters	
S_s – Acceleration Parameter	0.453
S_1 – Acceleration Parameter	0.226
F_a – Site Coefficient	1.3
F_v – Site Coefficient	*1.5*
S_{MS} – Adjusted MCE Spectral Response Acceleration Parameter	0.589
S_{M1} – Adjusted MCE Spectral Response Acceleration Parameter	*0.339*
S_{DS} – Design Spectral Response Acceleration Parameter	*0.393*
S_{D1} – Design Spectral Response Acceleration Parameter	*0.226*
PGA	0.193
PGA_M- MCE PGA adjusted for site effects	0.233
T_L – Long Period Transition Period	12

LIMITATIONS

This addendum report is subject to the “Risk Management” and “Limitations” sections of our April 10, 2018 report.

Please contact us if you have questions or require additional information.

Sincerely,

BLACKBURN CONSULTING

Robert C. Pickard, PG, CEG
Senior Engineering Geologist

Thomas W. Blackburn, GE, PE
Senior Principal

Copies: 1 to Addressee (PDF)

APPENDIX D.2

Geotechnical Design Report Update, Lincoln WWTRF Phase 1 and Phase 2 Expansion Project, Maturation Pond Pump Station, by Blackburn Consulting, June 2024

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File No. 3228.X
June 4, 2024

Mr. Gabe Aronow, P.E.
Stantec
3875 Atherton Road
Rocklin CA 95765

Subject: **GEOTECHNICAL DESIGN REPORT UPDATE, MATURATION POND PUMP STATION, REV 1**
Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
Placer County, California

Dear Mr. Aronow:

Blackburn Consulting is pleased to submit this Geotechnical Report Update letter for the proposed Lincoln Wastewater Treatment and Reclamation Facility (LWWTRF) Phase 1 and Phase 2 Expansion Project located in Placer County, California.

This addendum updates our April 10, 2018, Geotechnical Design Report recommendations for the Geotechnical Design Report for the Maturation Pond Pump Station. We understand the proposed type and location of improvements has not changed since our original report. We still consider our previous report recommendations appropriate unless specifically modified in this addendum.

SCOPE

To prepare this addendum, Blackburn reviewed our April 10, 2018, Geotechnical Design Report for the LWWTRF Phase 1 and 2 Expansion Project Maturation Pond Pump Station and updated the seismic design parameters.

UPDATED RECOMMENDATIONS

2022 California Building Code Seismic Parameters

Blackburn used the following to update the seismic (CBC) design parameters:

- SEAOC/OSHPD Seismic Design Maps Tool
- ASCE 7-16 Reference Standard
- Risk Category 2
- Site Class C – Stiff Soil
- Latitude: 38.859254 Longitude: -121.354847

We selected these inputs based on the subsurface conditions below the levee encountered in the boring and measured blow counts, and. Table 4 presents our updated 2022 CBC seismic design parameters.

Table 1: 2022 CBC Seismic Design Parameters	
S_s – Acceleration Parameter	0.455
S_1 – Acceleration Parameter	0.226
F_a – Site Coefficient	1.3
F_v – Site Coefficient	*1.5*
S_{MS} – Adjusted MCE Spectral Response Acceleration Parameter	0.592
S_{M1} – Adjusted MCE Spectral Response Acceleration Parameter	*0.340*
S_{DS} – Design Spectral Response Acceleration Parameter	*0.395*
S_{D1} – Design Spectral Response Acceleration Parameter	*0.226*
PGA	0.194
PGA_M- MCE PGA adjusted for site effects	0.234
T_L – Long Period Transition Period	12

LIMITATIONS

This addendum report is subject to the "Risk Management" and "Limitations" sections of our April 10, 2018 report.

Please contact us if you have questions or require additional information.

Sincerely,

BLACKBURN CONSULTING

PROFESSIONAL GEOLOGIST ROBERT C. PICKARD No. 2508 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER THOMAS W. BLACKBURN No. 2311 Exp. GEOTECHNICAL STATE OF CALIFORNIA

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Thomas W. Blackburn, GE, PE
Senior Principal

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APPENDIX D.3

Geotechnical Design Report Update, Lincoln WWTRF Phase 1 and Phase 2 Expansion Project, by Blackburn Consulting, April 2018

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
Placer County, CA

Prepared by:

BLACKBURN CONSULTING
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(530) 887-1494

April 2018

Prepared for:

Stantec
3875 Atherton Road
Rocklin, CA 95765

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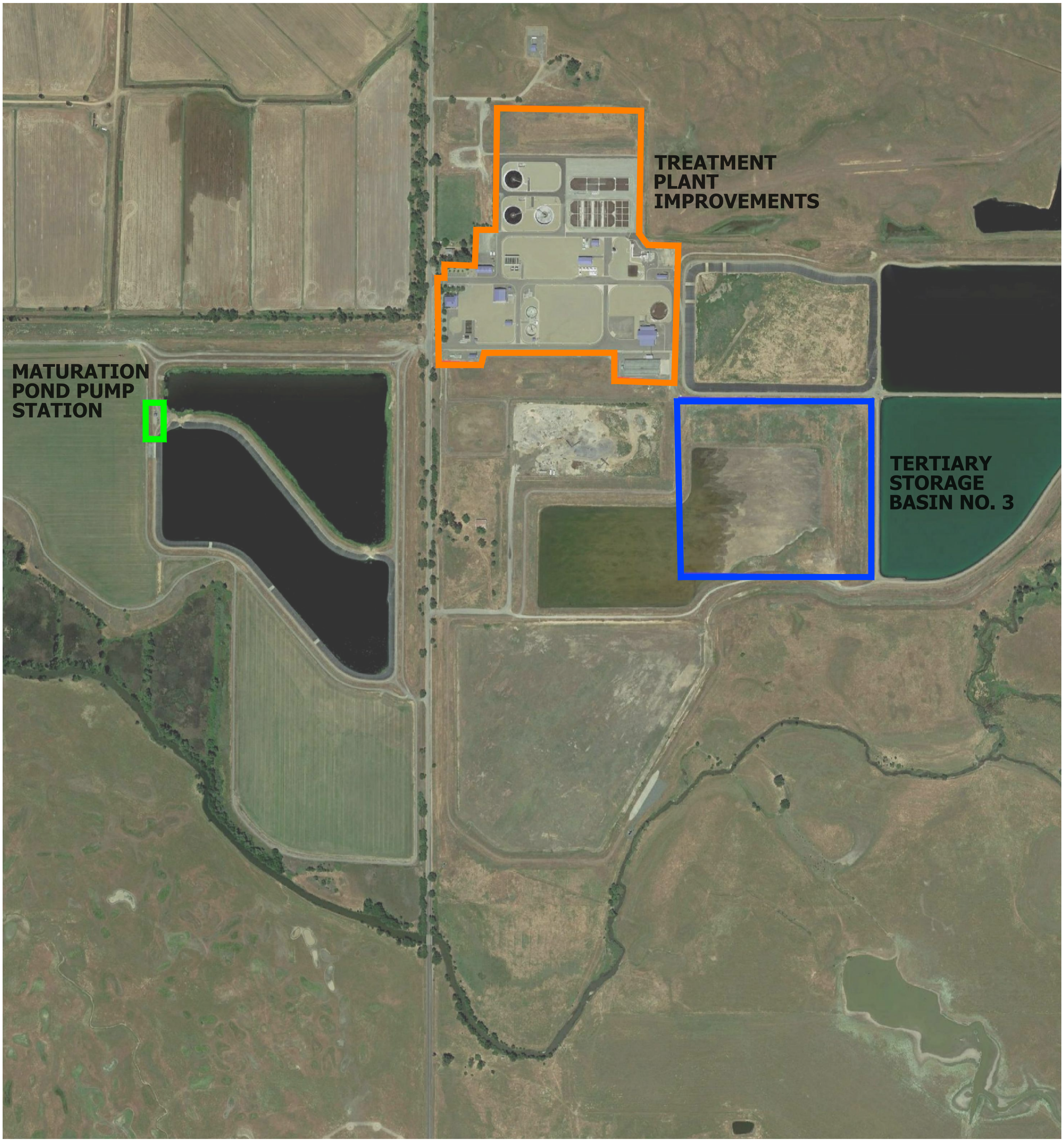
Phase 1 and Phase 2 Expansion Project
Placer County, CA



4/10/2018 3228.x Improvement Location Sheet.dwg

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IMPROVEMENT LOCATION SHEET

Lincoln Wastewater Treatment and Reclamation Facility
Placer County, California

File No. 3228.x

April 2018

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
WWTP Improvements
Placer County, CA

Prepared by:

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April 2018

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Geotechnical ▪ Geo-Environmental ▪ Construction Services ▪ Forensics

File No. 3228.X
April 10, 2018

Mr. Gabe Aronow, P.E.
Stantec
3875 Atherton Road
Rocklin CA 95765

Subject: **GEOTECHNICAL DESIGN REPORT**
Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
WWTP Improvements
Placer County, California

Dear Mr. Aronow:

Blackburn Consulting (BCI) is pleased to submit this Geotechnical Design Report for the Lincoln Wastewater Treatment and Reclamation Facility Phase 1 and Phase 2 Expansion Project, WWTP Improvements located in Placer County, California. BCI prepared this report in accordance with our June 6, 2017 agreement.

This report presents geotechnical and geologic data, and provides recommendations to design and construct the new facilities.

Please call us if you have questions or require additional information.

Sincerely,

BLACKBURN CONSULTING

PROFESSIONAL GEOLOGIST ROBERT C. PICKARD No. 2508 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER THOMAS W. BLACKBURN No. 2311 Exp. GEOTECHNICAL STATE OF CALIFORNIA

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GEOTECHNICAL DESIGN REPORT
Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
WWTP Improvements
Placer County, CA

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1 INTRODUCTION

1.1 Purpose

Blackburn Consulting (BCI) prepared this Geotechnical Design Report for an expansion to the City of Lincoln Wastewater Treatment and Reclamation Facility located in Placer County, California. This report presents geotechnical and geologic data and provides recommendations to design and construct the WWTP new support facilities included in the Phase 1 and Phase 2 Expansion Project.

We are aware pf the following geotechnical investigations on this site:

- 8/30/99 "Remote Storage Basins, East of Fiddyment Road, Placer County, California" by Carlton Engineering
- 3/5/2001 "Geotechnical Investigation Report" by Kleinfelder
- 1/31/2002 "Updated Geotechnical Investigation Report" by Kleinfelder
- 4/29/2013 "Geotechnical Design Report, Mid-Western Placer Regional Sewer Project" by BCI
- 11/27/2017 "Geotechnical Design Report, Lincoln Wastewater Treatment and Reclamation Facility Phase 1 and 2 Expansion Project" by BCI. This report updates and supersedes our 4/29/2013 report.

BCI prepared this report for Stantec to use during design and construction of the proposed improvements. Do not rely upon this report for different locations or improvements without the written consent of BCI.

1.2 Scope of Services

To prepare this report, BCI:

- Discussed the expansion improvements with Stantec
- Reviewed published geologic mapping, geotechnical information previously obtained for the project, and available geotechnical reports for existing facilities
- Reviewed and updated our engineering analysis and calculations

1.3 Site Location and Description

The expansion proposed project is located in an unincorporated area of Placer County. Figure 1 shows the project location.

The project consists of improvements at the City of Lincoln Wastewater Treatment and Reclamation Facility (WWTRF), as shown on Figure 2.

1.4 Project Description

We list the significant structural improvements included in the Phase 1 and 2 Expansion Project are listed in Table 1, below.

TABLE 1

Planned Structure	Approximate Plan Dimensions	Approximate Foundation Depth below grade
Grit Removal, basin and channels	Varied	10 ft
Oxidation Ditch	340 ft x 78 ft	22 ft
Oxidation Ditch Pump Station	18 ft x 21 ft	8 ft
Secondary Clarifier	110 ft diameter	23-38 ft
Dissolved air flotation system (DAFS)	64 ft diameter	17 ft to 26 ft
DAF Splitter	33 ft x 14 ft	16 ft
DAF Pump Station	9 ft diameter with 10.5 x 10.5 ft bottom slab	19 ft
Tertiary Filter Cell	59 ft x 33 ft	3 ft to 8ft

BCI will address the new tertiary storage basin and the new maturation pond outlet pump station in separate reports.

2 GEOLOGIC CONDITIONS

2.1 General Geology

Our site work and published geologic mapping[1] show the site is underlain by Quaternary deposits of the Riverbank Formation. Our borings confirm that the site is underlain by interbedded clays, silts, and sands.

The Riverbank Formation is an alluvial deposit typically composed of interbedded medium dense to dense sands, often cemented, and stiff to hard silts and clays. Bedding is typically horizontal, lenticular, and discontinuous. These sediments were deposited in the Late Pleistocene age (deposited over 150,000 years ago). This unit is shown as “Qrl” and “Qru” (Lower and Upper Riverbank) on Figure 3.

[1] Helley, E.J. and Harwood, D.S., 1985, Geologic Map of the Late Cenozoic Deposits of the Sacramento Valley and Northern Sierra Foothills: U.S. Geological Survey, Map MF-1790.

2.2 Faulting

The Fault Activity Map of California[2] does not identify Historic or Holocene age faults (displacement within the last 11,700 years) within or adjacent to the project site. The nearest mapped fault is the Cleveland Hill Fault located approximately 40 miles north of the site. Figure 4 shows the approximate location of faulting in the region.

3 FIELDWORK AND LABORATORY TESTS

3.1 Exploratory Borings

To characterize the subsurface conditions, BCI drilled, logged, and sampled 7 borings (LWWTRF-1 through LWWTRF-7) on September 24 and 25, 2012. Boring depth ranged from 21.5 to 51.3 feet below existing ground surface. Figure 2 shows the approximate boring locations. We include boring logs in Appendix A.

We located exploration points with a handheld GPS and using geographic features shown on the project topographic mapping. We did not survey the exploration points.

Our subcontractor, Taber Drilling, drilled the borings using 4-inch solid-stem auger and rotary wash techniques. We obtained soil samples at various intervals using a 3.0-inch O.D. Modified California (MC) sampler (equipped with 2.4-inch diameter brass liners), driven with an automatic hammer, weighing 140-pounds and falling approximately 30 inches.

A BCI geologist logged the borings and retrieved samples for laboratory testing. We used plastic caps to seal and label the 2.4-inch diameter, 6-inch long brass tubes retrieved from MC sampling. We also retrieved bulk soil samples from auger cuttings at varied depths, placed this material in large cloth bags, and labeled for laboratory identification.

During our field exploration, we performed field strength testing with a pocket penetrometer on select cohesive and/or cemented soil samples. We note the results of field tests on the boring logs.

3.2 Laboratory Testing

We completed the following laboratory tests on representative soil samples from our exploratory borings:

- Moisture content and unit weight for soil classification and in-place soil characteristics
- Plasticity index for soil classification and correlations
- Sieve analysis for soil classification and correlations

[2] Jennings, Charles W., and Bryant, William A., 2010 Fault Activity Map of California: California Geological Survey, Geologic Data Map No. 6.

- Unconfined compression for strength
- Maximum dry density for compaction characteristics
- Soil corrosivity (pH, minimum resistivity, chlorides and sulfates) performed by Sunland Analytical Laboratories for soil corrosion characteristics

We attach a laboratory summary sheet and laboratory test results in Appendix B and show test results on the boring logs.

4 SUBSURFACE FINDINGS

4.1 Soil Conditions

We encountered the following soil profile in our borings:

- Stiff to hard lean clays, lean clays with sand, and sandy lean clays with occasional dense clayey sands to depths of approximately 6 to 16 feet below ground surface (bgs)
- Interbedded layers of medium to very dense silty sands and clayey sands with stiff to hard lean clays and clean clays with sand to depths of approximately 18 to 23 feet bgs
- Very stiff to hard lean clays, lean clays with sand, and sandy clays to depths of approximately 38 to 41 feet bgs or to the base of the shallowest three explorations
- Dense and weakly cemented silty sand to the maximum depth explored (51.3 feet bgs)

Pocket penetrometer tests recorded on fine-grained soil samples retrieved from the borings were consistently at or above 4.0 tons per square foot (tsf), and unconfined compressive strengths test measured from 1.9 to 4.5 tsf, indicating relatively high compressive strengths. The silty sands have fines that are cohesive and/or are weakly to moderately cemented. Pocket penetrometer tests that we recorded on the silty sands were at or above 3.75 tsf and unconfined compressive strength tests measured 2.6 and 3.4 tsf.

Refer to the boring logs (Appendix A) for more specific subsurface conditions.

4.2 Groundwater

During our field exploration we encountered groundwater at the locations and depths listed in Table 2:

TABLE 2

Groundwater Summary	
Boring/Approximate Elevation (ft)	**Depth to Water/Approximate Elevation (ft)**
LWWTRF-1/110.5	23.9/86.9
LWWTRF-2/110.5	22.3/88.2
LWWTRF-3/110.5	26.5/84.0
LWWTRF-4/110.5	28.0/82.5
LWWTRF-5/110.5	27.1/83.4
LWWTRF-7/110.5	22.9/87.6

Groundwater has previously been recorded at shallower depths than what is shown above. Kleinfelder[3] recorded groundwater in their borings at depths ranging from 11.5 to 28.5 feet bgs (approximate elevations of 99 ft to 82 ft) in March-April 2000. A monitoring well placed by Kleinfelder, B-8, near the headworks, showed groundwater depths ranging from 13.0 ft in March 2000 to 16.9 feet in January 2001 (approximate elevations of 97.5 ft and 93.6 ft). It is not unusual to encounter channel sand lenses which can contain perched groundwater at varied depths within the Riverbank Formation. We also reviewed the Western Placer County Water Supply Appraisal[4], which shows regional groundwater elevations near 50 ft.

For project design, assume the highest groundwater elevation observed which is at a depth of 11.5 feet (approximately elevation 99 ft).

5 CONCLUSIONS AND RECOMMENDATIONS

The site will be suitable for the planned facilities when constructed in accordance with the project plans, industry standards, and our geotechnical recommendations. Some of the more significant site limitations include the presence of clay soils that will not be suitable for wall backfill, and relatively shallow groundwater that will require dewatering for some structure installations.

[3] Kleinfelder, 2002, Updated Geotechnical Investigation Report, Proposed Lincoln Wastewater Treatment Plant, Fiddyment Road, Placer County, California; consultant's report to Del Webb California Corporation
[4] Boyle Engineering, Western Placer County Water Supply Appraisal, Groundwater Elevations, Spring 1987.

5.1 Geologic Hazards

- Faulting—The potential for surface rupture or creep due to faulting at the site is very low. The Fault Activity Map of California[5] and the Geologic Map of the Sacramento Quadrangle[6] does not identify Historic or Holocene age faults (displacement within the last 11,700 years) within or immediately adjacent to the site. The site does not lie within or adjacent to an Alquist–Priolo Earthquake Fault Zone[7].
- Ground Shaking—The USGS, Earthquake Hazards Program, Seismic Design Maps (https://earthquake.usgs.gov/designmaps/us/application.php) indicate that for the design seismic event, a peak horizontal ground acceleration (PGA) of approximately 0.171g could be expected.
- Liquefaction—Our investigation shows a soil profile that consists of stiff to hard clays and medium dense to dense silty and clayey sands that are not liquefiable. Therefore, the potential for damaging liquefaction at the site is very low.
- Landslides and Slope Stability—Due to the relatively low topographic relief we do not expect landslides or natural slope failure.
- Seismically Induced Settlement—During a seismic event, ground shaking can cause densification of granular soil that can result in settlement of the ground surface. Considering the cohesive soils and medium dense soils observed in the borings, we consider the potential for significant seismically induced settlement to be very low.

5.1 Seismic Design

The project site is underlain by dense/very stiff to hard soils which is considered as Site Class C in the California Building Code (CBC).[8]

[5] Jennings, Charles W., and Bryant, William A., 2010 Fault Activity Map of California: California Geological Survey, Geologic Data Map No. 6.
[6] Wagner, D.L., et al, 1981, Geologic map of the Sacramento quadrangle, California, 1: 250,000: California Division of Mines and Geology, Regional Geologic Map 1A, scale 1: 250,000.
[7] Bryant, W.A., and Hart, E.W., 2007 (Interim Revision), Fault-Rupture Hazard Zones in California: California Department of Conservation, Division of Mines and Geology, Special Publication 42.

[8] California Building Code, 2016, California Code of Regulations, Title 24, Part 2 (Volume 2); published by International Conference of Building Officials and the California Building Standards Commission.

For seismic design of plant components, use the values in Table 3:

TABLE 3

CBC Seismic Design Parameters[9] (Site Class C)	
S_s – Acceleration Parameter	0.513 g
S_1 – Acceleration Parameter	0.253g
F_a – Site Coefficient	1.195
F_v – Site Coefficient	1.547
S_{MS} – MCE* Spectral Response Acceleration, Short Period	0.613 g
S_{M1} – MCE* Spectral Response Acceleration, 1-Second Period	0.391 g
S_{DS} – 5% Damped Design Spectral Response Acceleration, Short Period	0.408 g
S_{D1} – 5% Damped Design Spectral Response Acceleration, 1-Second	0.261 g
T_L – Long Period Design Period**	12 seconds
PGA – Peak Ground Acceleration	0.171 g
PGA_M – Site Modified Peak Ground Acceleration	0.206 g

* Maximum Considered Earthquake
** Figure 22-12, ASCE 7-10

5.2 General Grading Recommendations

5.2.1 *Excavation Conditions*

Based on the soil conditions and drilling performance, excavation is possible with conventional equipment (common earthmoving equipment and large backhoe/excavator). The fine-grained and hard soil conditions can create slow excavation conditions.

5.2.2 *Site Clearing*

Prior to trenching or making any cuts and fills, remove all debris, trees and brush including the root system and strip surface vegetation to a depth of 4 inches below the surface. Excavations resulting from trees, brush, and debris removal should be deepened and widened to provide access to self-propelled compaction equipment. Remove strippings from the site or use as landscape soil in designated areas.

[9] California Building Code, 2016, California Code of Regulations, Title 24, Part 2 (Volume 2); published by International Conference of Building Officials and the California Building Standards Commission.

5.2.3 *Original Ground and Subgrade Preparation*

Process and compact the exposed soil in at-grade, cut, and fill areas as follows:

- Scarify the exposed soil to a depth of approximately 8 inches.
- Moisture condition subgrade to within 3% of the optimum moisture content.
- Compact the subgrade soil to a minimum 90% relative compaction based on ASTM D1557

Where fill will be placed on or against slopes with a gradient of 5(H):1(V) or steeper, fill must be benched into the slope. Benching must remove loose surficial soils and result in stepped benches, generally one to two feet in height and depth into the existing slope. Where benching will interfere with existing structures, utilities, or vegetation, BCI can review modifications and on a case-by-case basis.

For fills that are 5 feet or higher and placed on or against a slope with a gradient of 5:1 or steeper, provide a key at the toe of the fill slope. The key must be a minimum of 10 feet wide, one foot deep, sloped a minimum of 2% into the slope, and extend 2 feet beyond the fill toe. Where restricted access will not allow for a toe-bench 10 feet wide, the bench can be reduced to a minimum width of 6 feet provided the fill slope is less than 10 feet in height and the contractor can show that compaction equipment can achieve the specified compaction for the full width of the bench.

5.2.4 *General Fill Placement and Compaction*

General fill (**not trench or structure backfill**) may consist of on-site soil provided it contains no rocks larger than 4 inches in maximum dimension. Fill should be free of debris and concentrations of vegetation.

If import for general fill is required, it must meet the following requirements:

- Classified as Silt (ML), Silty Sand (SM), Silty Gravel (GM),

General Backfill Import Requirements			
Gradation		**Test Procedures**	
Sieve Size	**Percent Passing**	**ASTM**	**Caltrans**
3 inch	100	D6913	202
No. 200	20-70	D6913	202
Organic Content			
Less than 3%		D2974	
Expansion Index			
Less than 20		D4829	

- Approved by BCI prior to site delivery.

Place and compact fill as follows:

- Place fill in maximum 8-inch-thick loose lifts,
- Moisture condition the soil within 3% of optimum
- Compact the soil to a minimum 90% relative compaction based on ASTM D1557.

Test all fill at vertical increments of not more than 1 foot and at final grade or pavement subgrade. For horizontal testing frequency, use the following minimums:

- One test for every 100 square feet around structures
- One test for every 500 square feet for structure pads

Complete at least one compaction curve (Proctor) for each material type, source location (for import), and as changes in native materials occur. Material changes include a change in material designation based on the Unified Soil Classification System.

5.2.5 *Fill Slopes*

Construct fill slopes no steeper than 2(H):1(V). To achieve adequate compaction on the face of fill slopes, over-build the slopes and then cut back to the design grade. Track-walking is not an adequate method to compact the face of slopes.

5.3 Dewatering

Dewatering may be required for installations greater than approximately 11 feet deep (see Section 4.2). Significant groundwater inflow should be anticipated at the deeper excavations such as for the oxidation ditch, secondary clarifier, DAFS, DAF splitter, and DAF Pump Station.

Dewatering can consist of:

- Deep sumps within the excavation. Considering the presence of fine-grained soils and relatively flat lying bedding, sumps within the excavation are not likely to provide good drawdown.
- Well points. Well points will likely work better to cut off flow into the excavation and drawdown the water level over a larger area.

To facilitate work at the base of the excavation, groundwater should be drawn down at least 5 feet below the planned bottom of excavation. The need for dewatering can be reduced by planning excavations during the lowest anticipated seasonal water levels (expected during the late summer and fall months).

5.4 Temporary Excavations

Temporary excavations will require sloping and/or shoring in accordance with Cal OSHA requirements. Based on our subsurface exploration and laboratory testing, preliminary excavation and shoring design may be based on Type A soil to planned excavation depth. For Type A soil conditions, temporary excavations may be sloped at ¾(H):1(V).
Where groundwater is present or cohesionless/uncemented granular soils are encountered, Type C soil conditions will apply and a 1.5(H):1(V) slope gradient is required.

The impact of existing structures, traffic vibrations, actual soil conditions exposed in the open trenches, and other factors that may promote trench wall instability must be evaluated at the time of construction and trench sloping/shoring adjusted accordingly. Surcharge loads such as trench spoils, equipment, etc. should not be placed adjacent to an open excavation (within a distance of ½ the height of the trench). ***The above is guideline information only.***
The contractor is responsible for the safety of all excavations and should provide appropriate excavation sloping and shoring in accordance with current Cal OSHA requirements and observe conditions observed during construction for necessary modification and safety.

5.5 Foundation Design

5.5.1 *At-Grade Shallow Foundations*

If the designers and contractors follow our grading and construction recommendations below, foundations for structures such as the tertiary filter cell can consist of shallow strip footings and isolated spread footings. We expect footings for at-grade structures to be founded on compacted fill and/or firm native soils.

- Embed continuous strip and isolated footings a minimum of 18 inches into the lowest adjacent prepared subgrade.
- Both strip and isolated footings must be a minimum of 18 inches wide. Size strip and isolated footings not to exceed an allowable bearing capacity of 3,000 pounds per square foot (dead load plus live load). The allowable bearing capacity may be increased by one-third if seismic and/or wind loads are included.
- Total settlement is expected to be less than ¾-inch and differential settlement less than ½-inch over a length of 50 feet.
- To resist lateral movement, use a coefficient of friction of 0.40 psf at the base of the foundation and a passive earth pressure of 300 psf per foot of embedment depth up to a maximum of 3,000 psf. Ignore the upper one-foot of footing depth (below the lowest adjacent soil grade) in determination of the passive pressure. Both frictional resistance and passive earth pressure can be combined for lateral resistance; when combined, increase the safety factor against sliding from a minimum of 1.5 to 2.0.
- Concrete slabs with crushed rock underlayment may be designed using a Modulus of Subgrade Reaction, k_s, of 150 pounds per cubic inch (pci) in cut or fill locations where engineered fill is placed as recommended in this report.

- Clean footing excavations of debris and loose soil prior to placing concrete.
- BCI must observe all footing excavations prior to reinforcement placement to verify competent bearing materials.
- Slope the ground surface away from foundations at a minimum of 2 percent for a distance of at least 5 feet.

5.5.2 *Below-Grade Foundations*

5.5.2.1 Bearing Capacity

Most of the planned structures listed in Table 1 are substantially below-grade structures. For these structures, the net pressure exerted upon the subsurface will be similar to or less than the current load. Excavation for below-grade structures reduces the net pressure by removing soil that acts as a "preload" to the underlying soils, thus "unloading" the bearing materials before "loading" by placement of the structure.

Below grade structures will use mat type foundations for support. For structures at depths greater than 8 feet:

- Use a maximum net contact pressure of 3,500 psf.
- Use a Modulus of Subgrade Reaction, k_s, equal to 200 pci.
- We expect settlement of mat foundations is expected to be less than 1 inch with differential settlement less than ½-inch over a distance of approximately 100 feet.
- Clean footing excavations of debris and loose soil prior to placing concrete.
- BCI must observe all footing excavations prior to reinforcement placement to verify competent bearing materials.
- For ground preparation and subgrade uniformity, Class 2 aggregate baserock can be used as underlayment (this is not geotechnically necessary provided a firm uniform subgrade is obtained). If an aggregate underlayment is used, place a minimum thickness of 6-inches and compact to a minimum of 95% relative compaction (per ASTM D 1557 test method).
- Crushed rock underlayment may also be used (and can benefit excavation dewatering). Envelope the crushed rock with a geotextile filter fabric (ie. Mirafi 140N) and compact the rock with a static roller.

If isolated spread footings or piers are required for column support, BCI can provide additional recommendations when the planned design and approximate loading is available.

5.5.2.2 Structure Backfill

Native soils consist predominately of lean clay which will not be suitable for structure backfill. The contractor may import structure backfill or lime treat native soils.

BCI must approve import structure backfill prior to delivery. Use the specifications in Table 4 for import structure backfill for all below-grade structures:

TABLE 4

Import Structure Backfill Requirements			
Gradation		**Test Procedures**	
Sieve Size	**Percent Passing**	**ASTM**	**Caltrans**
3 inch	100	D6913	202
¾ inch	70-100	D6913	202
No. 4	50-100	D6913	202
No. 200	0-50	D6913	202
Plasticity			
Plasticity Index	<15	D4318	204
Organic Content			
Less than 3%		D2974	
Expansion Index			
Less than 20		D4829	

Prior to placement of lime treated soil as structure backfill the contractor must:

- Perform lab testing to sufficiently determine the percentage of lime needed to meet specifications. Retain BCI to provide concurrent quality control tests and approve proposed percentage of lime to be used.
- Provide written means and methods of lime treatment.

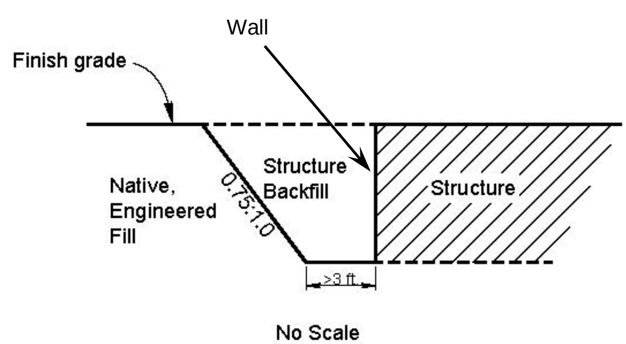
BCI must observe mixing of lime with soil.

Based on previous experience at the site, we recommend 3% lime for preliminary planning and bidding purposes. Use the specifications in Table 5 for lime treated structure backfill requirements.

TABLE 5

Lime Treated Structure Backfill Requirements			
Gradation		**Test Procedures**	
Sieve Size	**Percent Passing**	**ASTM**	**Caltrans**
3 inch	100	D6913	202
¾ inch	70-100	D6913	202
No. 4	50-100	D6913	202
No. 200	20-70	D6913	202
Plasticity			
Plasticity Index	<12	D4318	204
Organic Content			
Less than 3%		D2974	
Expansion Index			
Less than 20		D4829	

As shown below, the zone of placement for structure backfill should extend up from the base of the wall at a slope of 0.75(H):1(V) and at least 3 feet behind the wall. Native, engineered fill may be placed beyond the structure backfill zone.



- Moisture condition backfill to within 2% of optimum and place in maximum 8-inch thick, horizontal, loose lifts.
- Compact backfill to a minimum 92% relative compaction based on the ASTM D 1557 test method.

To minimize the residual lateral earth pressures on structure walls compaction equipment used behind the walls must be restricted (by load and distance from wall) so that wall design values are not exceeded. We recommend compaction within a horizontal distance equal to one-half of the wall height (to a maximum distance of 5 feet), be completed with hand-operated equipment (i.e., jumping jack).

To minimize the potential for significant settlement around deep walls, controlled low strength material (CLSM) can be used to backfill to the surface or to a manageable depth (e.g. 10 feet below grade).

5.5.2.3 Lateral Earth Pressures

The below grade structures will act as retaining structures. Walls will retain compacted select imported soils meeting the requirement for structure backfill. For evaluation of lateral earth pressures, use the equivalent fluid weights (EFW) shown below in Table 6. We show values for both drained and undrained backfill with level ground conditions; the drained condition assumes groundwater cannot accumulate behind the wall (backfill is drained).

TABLE 6

LATERAL EARTH PRESSURES		
Condition	**Equivalent Fluid Weight (pcf)**	
	Drained	**Undrained**
At-Rest	62	95
Active	40	84
Passive	300	160
Seismic (Active and At-Rest)	6	6

The above pressures assume structure backfill placed against the structure wall in accordance with our recommendations, a saturated (total) unit weight of approximately 135 pounds per cubic foot (pcf) and a minimum internal angle of friction of 32 degrees. Notify BCI if these assumptions are not valid so that we may assess the situation and provide additional recommendations, if necessary. Backfill with CLSM is an acceptable alternative.

For seismic loading, add the Seismic EFW to the at-rest or active EFW weight and apply the total force as a uniform load on the wall with a resultant located at 0.5H where H is the backfill height. We estimated the EFWs for seismic loading using the Mononobe-Okabe equation and a horizontal seismic acceleration coefficient, k_h, of approximately ½ the expected PGA. This k_h value assumes that the walls displace at least 1-inch during the design seismic event.

Surface loads (footings, storage, vehicle traffic) applied near the wall will increase the lateral pressure on the wall. A uniform surface load of 200 psf to 300 psf is often used to approximate construction traffic loading on walls. In general, if surface loads are closer to the edge of the retaining wall than three-fourths of the retained height, increase the design wall pressure by 0.5q over the area of the retaining wall. In this expression, q is the surface surcharge load in psf. This is a conservative procedure and lower design pressures may be applicable upon evaluation of individual surface loads and setback distances.

For drained conditions, provide adequate drainage to avoid build-up of hydrostatic pressures. Positive drainage for retaining walls should consist of a vertical layer of permeable material, such as a graded sand and gravel (graded to meet Caltrans Standard Specifications for Class 1, Type A Permeable Material), pea gravel, or crushed rock, at least 6 inches thick, positioned between the retaining wall and the backfill.

If pea gravel or crushed rock is used, place a nonwoven filter fabric between it and the backfill to prevent the drain from becoming clogged. A synthetic drainage fabric, such as Enkadrain (Colbond Geosynthetics Co.), Miradrain (TC Mirafi) or an equivalent, may be substituted for the permeable layer. Use care during installation to assure that the filter part of the material faces the backfill. Remove collected water by installing weep holes along the bottom of the wall or by a perforated drainage pipe along the bottom of the permeable material or drainage fabric continuously sloped towards suitable drainage facilities (i.e., gravity drain or sump pump).

5.5.2.4 Buoyancy Resistance

As discussed in section 4.2, groundwater may occur at depths as shallow as 11 feet bgs. In undrained conditions, below grade structures may be subjected to an uplift load (buoyancy). The uplift force will be resisted by the weight of the structure and the weight of the backfill overlying foundation extensions (if any).

If Stantec designs foundation extensions, calculate the resistance against uplift due to the weight of the soil, use a backfill unit weight of 130 pcf above groundwater and 73 pcf below groundwater, with a soil wedge extending up from foundation extensions at an angle of 30 degrees from vertical.

Frictional resistance from surrounding soils can be used to resist uplift as well. The frictional resistance will vary with depth but can be assumed as follows (apply a factor of safety of at least 2 to determine the allowable uplift resistance):

For structure backfill against a concrete structure:

- 24 psf per foot of depth where above the design groundwater level
- 13 psf per foot of depth when below the design groundwater level

For a vertical soil interface such as over a foundation extension:

- 38 psf per foot of depth where above the design groundwater level
- 21 psf per foot of depth when below the design groundwater level

Stantec has indicated they may use a system of Cast in Drilled Hole (CIDH) piles, likely with "belled" bottoms to resist uplift due to groundwater. Pile shafts are expected to be 2 feet in diameter. For the proposed piles, we provide the following options:

- Straight Shaft Pile (2-foot diameter):
    - Allowable uplift resistance: 2,100 pounds per foot of pile (ignore lower 2 feet).
- Belled Pile (2-foot diameter shaft):
    - Bell diameter: 5 feet
    - Minimum pile length: 14 feet (to bottom of bell)
    - Allowable uplift resistance: 60 tons (not including the weight of the pile)
- Belled Pile (2-foot diameter shaft):
    - Bell diameter: 4 feet
    - Minimum pile length: 10 feet (to bottom of bell)
    - Allowable uplift resistance: 30 tons (not including the weight of the pile)

5.5.2.5 Lateral Resistance

Lateral resistance for retaining structures can be achieved through friction and passive earth pressures. For design, use a coefficient of friction of 0.40 (below or above groundwater) at the base of the concrete footing and a passive earth pressure of 300 psf per foot of embedment depth. Passive earth pressures may be increased up to 400 psf per foot if lateral movements of up to 2% of the embedment depth can be tolerated. Limit passive earth pressures to a maximum of 3,000 psf (additional passive pressure can be evaluated for specific locations if necessary). Decrease the passive pressure to 160 psf when below design groundwater levels. Do not include the upper 1-foot of soil in passive resistance calculations. Where passive pressure or friction alone is used against sliding, use a minimum factor of safety of 1.5 for lateral stability (1.1 if seismic loading is included). Where both passive pressure and friction are used to resist sliding, use a minimum factor of safety of 2.0.

5.6 Minor Structures (Valve Vaults, Access Ways, etc.)

Provided that the recommendations in this report are followed, minor structures (such as valve or blow-off vaults, access ways, etc.) may be founded on concrete mat or strip footings, or a compacted granular base (minimum of 6 inches of Class 2 baserock) if appropriate.

- Embed the foundations a minimum of 18 inches below the lowest adjacent prepared subgrade into firm native soil or compacted fill/backfill.

- Footings must be a minimum of 12 inches wide and sized not to exceed an allowable bearing capacity of 3,000 psf. The allowable bearing capacity may be increased by one-third if seismic and/or wind loads are included.
- If additional bearing capacity is required for specific minor structures, we can review and provide recommendations on a case-by-case basis.
- To resist lateral movement, use a coefficient of friction of 0.40 at the base of the foundation and a passive earth pressure of 300 psf per foot of embedment depth up to a maximum of 3,000 psf. Ignore the upper one-foot of footing depth (below the lowest adjacent soil grade) in determination of the passive pressure. Both frictional resistance and passive earth pressure can be combined for lateral resistance; when combined, increase the safety factor against sliding from a minimum of 1.5 to 2.0.

If necessary for evaluation of lateral loading on shallow vaults, use an At-Rest equivalent fluid weight of 65 pcf for the drained condition and 95 pcf for undrained. The drained condition assumes groundwater does not accumulate; the undrained condition would be applied below an assumed groundwater level.

We based these values on foundations bearing on native soil and native soil backfill compacted against vault walls.

5.7 Soil Corrosivity

Our subcontractor, BSK, tested soil samples from our borings for corrosion characteristics (pH, resistivity, chlorides, and sulfates). We show the corrosion test results in Table 7.

TABLE 7

Laboratory Soil Corrosivity Results					
Boring/Trench Location	**Sample No./ Depth (ft)**	**pH**	**Minimum Resistivity (ohm-cm)**	**Chloride (mg/kg)**	**Sulfate (mg/kg)**
LWWTRF-1	Bag B/ 0.0-10.0	7.7	1,930	18	20
LWWTRF-5	5/ 25.0-26.5	7.5	1,040	24	8
LWWTRF-7	3/ 15.0-16.5	7.7	1,220	28	10

American Concrete Institute (ACI) 318 Table 4.3.1 provides guidance on concrete exposed to sulfate. Results of laboratory testing indicate a negligible sulfate exposure for the representative soil samples.

Caltrans considers a site to be corrosive if one or more of the following conditions exist for the representative soil samples taken at the site:

- Chloride concentrations greater than or equal to 500 parts per million (ppm),
- Sulfate concentration is greater than or equal to 2000 ppm, or
- pH is 5.5 or less.

Based on these test results, the site would be considered non-corrosive. However, the relatively low resistivity values and the presence of the fine-grained soils suggest the soil may be corrosive to metals. We recommend that a corrosion engineer review these results and provide corrosion mitigation recommendations.

5.8 Concrete Slabs on Grade

5.8.1 *Slab Underlayment*

Concrete slab-on-grade may be used provided the contractor(s) prepares the structure pads in accordance with our grading recommendations and any addenda by BCI. Underlay the concrete slabs with a minimum of 4 inches of washed, crushed, and compacted rock to provide uniform support. Grade crushed rock used beneath floor slabs such that 100% passes the ¾ inch sieve and less than 5% passes the No. 4 sieve. Compact crushed rock with at least two passes of a vibratory type compactor.

Exterior flatwork may be placed directly on the prepared subgrade without the use of rock underlayment. Subgrade must be free of debris, uniformly compacted, and thoroughly wetted before placing concrete.

5.8.2 *Slab Design*

Concrete slabs with crushed rock underlayment may be designed using a Modulus of Subgrade Reaction, k_s, of 150 pci in cut or fill locations where structural fill is placed as recommended in this report.

5.9 Trench Backfill and Compaction

5.9.1 *Pipe Bedding and Pipe Zone Material*

Support pipe on a minimum of 4 inches of granular bedding and in accordance with the pipe manufacturer's recommendations. Although we do not anticipate soft, unsuitable pipe subgrade at any particular location, it can occur with shallow groundwater conditions and sandy soils. Notify the project engineer and BCI for review and mitigation recommendations if encountered. To achieve a stable and non-yielding subgrade suitable for pipe placement and backfilling, typical mitigation may include:

- Replacement of unsuitable subgrade with ¾-inch minus crushed rock (minimum of 6 inches)
- Enclose rock in geotextile filtration fabric such as Mirafi 140N (or equivalent).

A granular pipe zone material may be used. Native soils will contain a significant amount of fines (passing #200 sieve) and will **not** be suitable for bedding or pipe zone backfill. For pipe bedding and initial backfill material (which extends to 1 foot above the top of pipe) use material that meet the specification in Table 8.

TABLE 8

Pipe Bedding and Initial Backfill Requirements			
Gradation		**Test Procedures**	
Sieve Size	**Percent Passing**	**ASTM**	**Caltrans**
1 inch	100	D6913	202
¾ inch	90-100	D6913	202
No. 4	35-60	D6913	202
No. 30	10-30	D6913	202
No. 200	2-5	D6913	202
Sand Equivalent			
Minimum 25		D2974	

BCI considers the following materials to be suitable as alternative pipe zone (bedding) backfill material:

- Controlled Low Strength Material (CLSM)
- Controlled Density Fill (CDF)

A modulus of soil reaction (E') of 4,000 psi can be used for granular pipe zone backfill if compacted to >90% relative compaction (ASTM D 1557).

5.9.2 *Trench Backfill*

Trench backfill (intermediate backfill) may consist of excavated soils. Fill should be free of debris and concentrations vegetation or clay soils and meet the specifications in Table 9.

TABLE 9

Intermediate Trench Backfill Requirements			
Gradation		**Test Procedures**	
Sieve Size	**Percent Passing**	**ASTM**	**Caltrans**
3 inch	100	D6913	202
No. 200	20-70	D6913	202
Organic Content			
Less than 3%		D2974	
Expansion Index			
Less than 20		D4829	

5.9.1 *Trench Backfill Compaction*

Follow the pipe manufacturer's requirements for initial backfill to avoid damage to the pipe. To facilitate compaction in the pipe zone area (top of bedding up to 12 inches above pipe), use a trench width that provides a minimum clearance of 12 inches between the pipe and trench wall.

- Moisture condition trench backfill to within 2% of optimum moisture content and compact to a minimum 92% relative compaction (based on ASTM 1557).
- Use a maximum compacted lift thickness of 8 inches unless field performance testing can demonstrate adequate compaction of thicker lifts.
- Jetting is not acceptable for compaction.

Test all trench backfill (bedding, pipe zone backfill, trench zone, etc.):

- At vertical increments of not more than 1 foot and at final grade or pavement subgrade.
- At horizontal testing frequencies of at least one test for every 200 linear feet of pipe (both sides of pipe in pipe zone).
- Complete at least one compaction curve (Proctor) for each material type, source location (for import), and as changes in native materials occur. Material changes include a change in material designation based on the Unified Soil Classification System.
- Testing frequency can be adjusted based on contractor performance, ease of compaction, and material variability.

Soil excavated during pipe installation can have moisture contents well over optimum, especially during the winter and spring months or if perched water is encountered. In this case, it will be necessary to dry back the soil to within 2% of optimum moisture content prior to use as backfill.

It is important to achieve compaction of pipe zone materials at the pipe haunches and spring line; compaction below the pipe spring line will be a difficult task for the contractor. We recommend a compaction demonstration section to test placement and compaction means and methods for each material type that will be used.

5.9.2 *Trench Backfill Settlement*

If pipeline backfill is placed, compacted, observed, and tested as recommended above, we expect potential settlement at the surface to be less than ½-inch (0.25% to 0.50% of backfill depth) for planned pipeline depths. The magnitude of surface settlement will be affected by the degree and uniformity of backfill compaction; therefore, it is important that backfill methods are observed and compaction checked at frequent intervals where limiting potential settlement is important. This is especially critical where the pipeline crosses beneath roadways and other utilities.

5.10 Hot Mix Asphalt (HMA) Pavement Design

New pavement may be planned at the project site. Kleinfelder[10] provided design recommendations for the existing pavement at the site. Kleinfelder obtained Resistance (R)-Values for the subgrade soils that range from 9 to 19 with most values in the range of 9 to 12. These R-Values are appropriate for the material types (lean clay to sandy clay) we observed at or near planned subgrade elevation. Stantec indicates that the existing pavement has performed well and there are no apparent deficiencies.

Use Table 10 for pavement design from Klienfelder's report dated January 31, 2002[11] and checked by BCI.

TABLE 10

R Value = 10		
Design Traffic Index	**Material Type/Depth Required**	
	Dense Graded Asphalt Concrete, inches	**Aggregate Baserock Class 2, inches**
5.5	3.0	11.5
7.5	4.5	15.5

[10] Kleinfelder, 2002, Updated Geotechnical Investigation Report, Proposed Lincoln Wastewater Treatment Plant, Fiddyment Road, Placer County, California; consultant's report to Del Webb California Corporation.
[11] Kleinfelder, 2002, Updated Geotechnical Investigation Report, Proposed Lincoln Wastewater Treatment Plant, Fiddyment Road, Placer County, California; consultant's report to Del Webb California Corporation.

5.10.1 *Pavement Subgrade Preparation*

To develop the pavement structural sections above, we assume that the native soils will be used as indicated for subgrade materials and the subgrade will be prepared, placed, and compacted as outlined below:

1. Strip vegetation, where applicable, to the maximum depth of the vegetative layer or a minimum depth of 4 inches bgs. Do not use strippings within engineered fill.
2. Scarify a minimum depth of 8 inches, moisture condition to near the optimum moisture content, and compact to a minimum of 90% relative compaction based on ASTM D 1557.
3. Check subgrade stability by running a loaded water truck over the subgrade. Mitigate unstable areas as recommended by BCI (see the options a through d, presented below).
4. Place and compact aggregate base (AB) to a minimum 95% relative compaction (ASTM D 1557).
5. Check AB stability under construction equipment. Mitigate unstable areas observed in the AB layer as recommended by BCI prior to placing asphalt.

Yielding subgrade soil conditions can typically be stabilized using one of the methods listed below; however, BCI and/or the project engineer should review soil conditions and approve mitigation methods prior to implementation.

a) Deep scarify and allow wet subgrade soils to air dry.
b) Remove wet soils to a firm base and allow the exposed soil to dry to near optimum moisture content and/or replace with drier soil.
c) Lime or cement treat to reduce the moisture content of subgrade soils.
d) Remove yielding soils to a firm base or 2 feet below subgrade elevation, whichever is less. Place a layer of stabilization fabric or grid (such as Mirafi 500X, Tensar BX1100, or an equivalent) and backfill the overexcavation with compacted Class 2 AB.

The long-term performance of the pavement is dependent upon:

1. Uniform and adequate compaction of the soil subgrade,
2. Adequate compaction of engineered fill and utility trench backfill beneath the pavement,
3. Positive drainage,
4. Limiting water under pavement with cut-offs at planter areas.

Perform earthwork within pavement areas in accordance with the recommendations contained within this report.

The design TIs used are assumed and the project civil engineer should select the appropriate TI based on the anticipated traffic frequency and load. BCI can provide structural sections based on additional TIs if necessary.

6 RISK MANAGEMENT

Our experience and that of our profession clearly indicates that the risks of costly design, construction, and maintenance problems can be significantly lowered by retaining the geotechnical engineer of record to provide additional services during design and construction.

For this project, we recommend that the project owner retain us to:

- Review and provide comments on the civil plans and specifications prior to construction.
- Monitor construction to check and document our report assumptions. At a minimum, BCI should observe foundation excavations, approve backfill, test backfill compaction, observe and test placement and compaction of fill for structures.
- Update this report if design changes occur, 2 years or more lapses between this report and construction, and/or site conditions have changed.

If we are not retained to perform the above applicable services, we are not responsible for any other party's interpretation of our report, and subsequent addendums, letters, and discussions.

7 LIMITATIONS

BCI performed services in accordance with generally accepted geotechnical engineering principles and practices currently used in this area. Where referenced, we used ASTM and California Test Method standards as a general (not strict) guideline only. Do not use or rely upon this report for different locations or improvements without the written consent of BCI. We do not warranty our services.
BCI based this report on the current site and alignment conditions. We assume the soil and groundwater conditions encountered in our explorations are representative of the subsurface conditions throughout the site. Conditions at locations other than our explorations could be different.

Logs of our explorations are presented in Appendix A. The lines designating the interface between soil types are approximate. The transition between material types may be abrupt or gradual. Our recommendations are based on the final logs, which represents our interpretation of the field log and general knowledge of the site and geological conditions. Soil and rock descriptions on the boring and test pit logs are based on our field logging, geologic mapping, seismic refraction surveys, and laboratory testing.

The groundwater elevations discussed in this report represent the groundwater elevation during the time of our subsurface exploration, at the specific exploration locations, and groundwater observed by others. The groundwater table may be lower or higher in the future and at other locations.

Modern design and construction are complex, with many regulatory sources/restrictions, involved parties, construction alternatives, etc. It is common to experience changes and delays. The owner should set aside a reasonable contingency fund based on complexities and cost estimates to cover changes and delays.

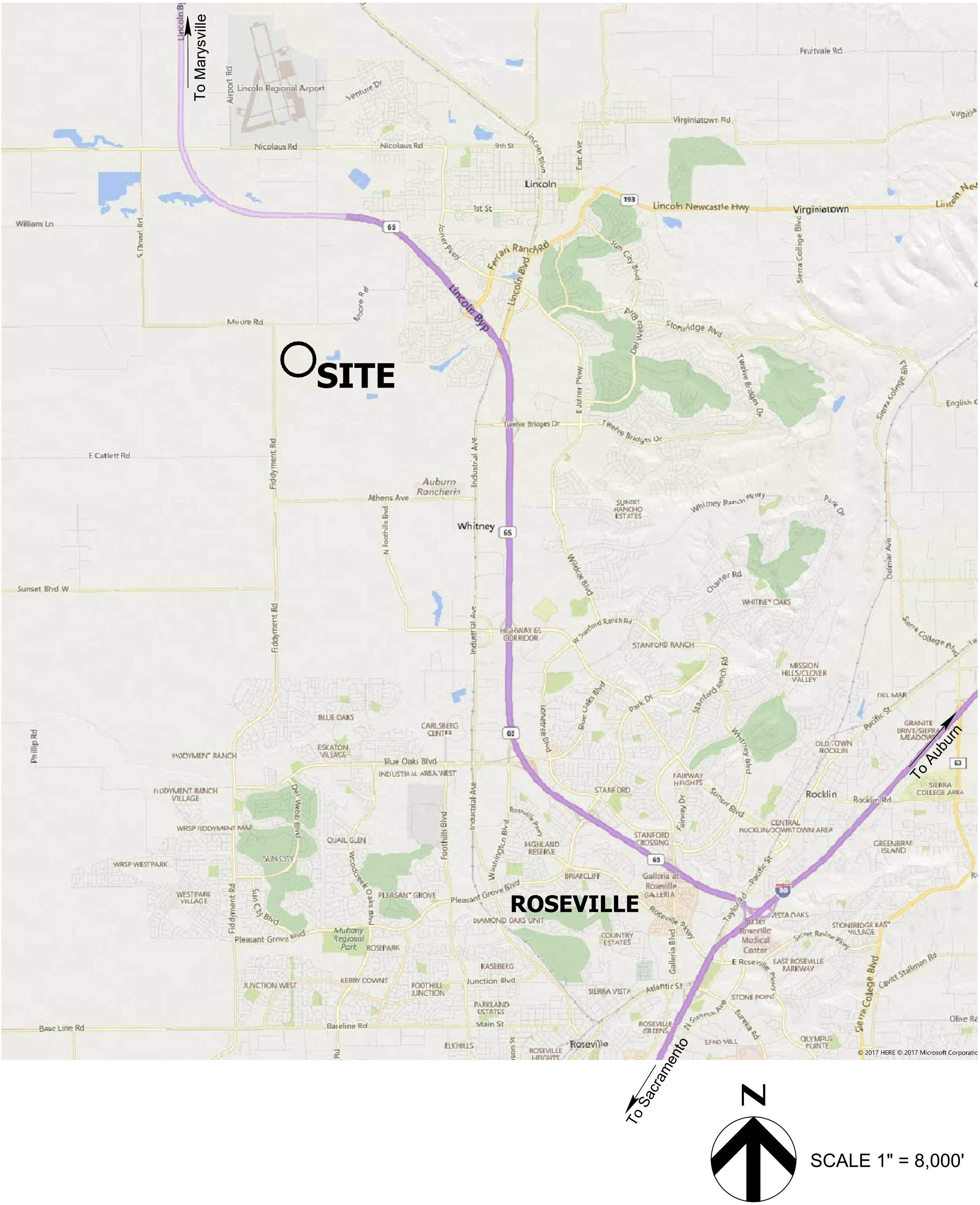
We include guidelines for using this report in Appendix C.

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
WWTP Improvements
Placer County, CA

FIGURES

Vicinity Map
Site Map
Regional Geologic Map
Regional Fault Map



SCALE 1" = 8,000'

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VICINITY MAP

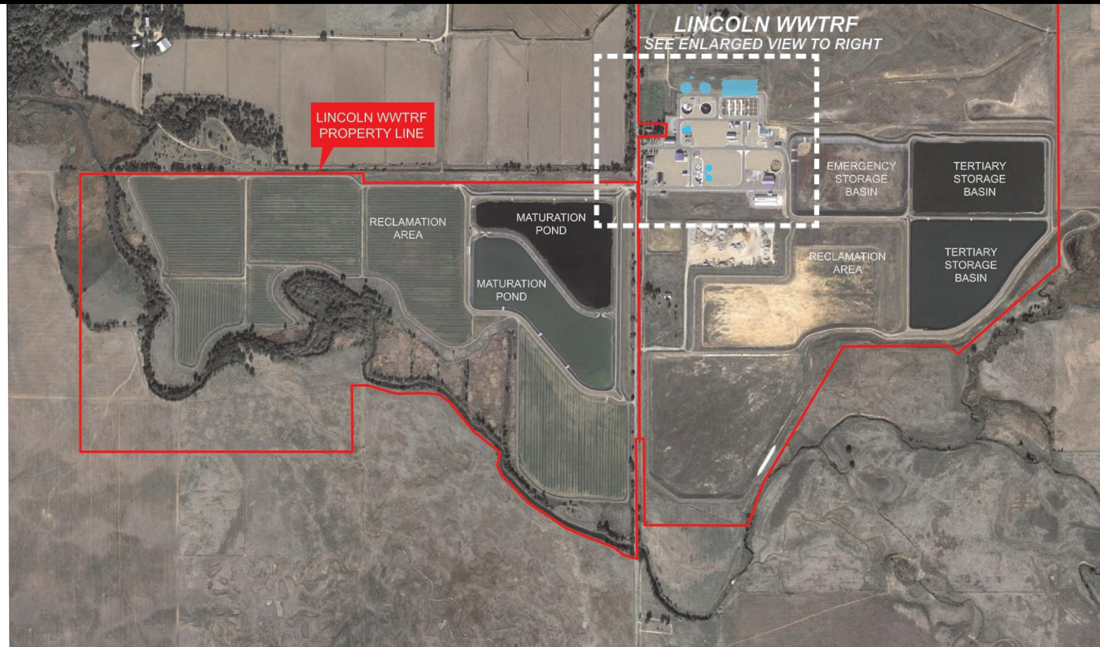
Lincoln Wastewater Treatment and Reclamation Facility, Phase 1 and Phase 2 Expansion Project
Placer County, California

File No. 3228.x

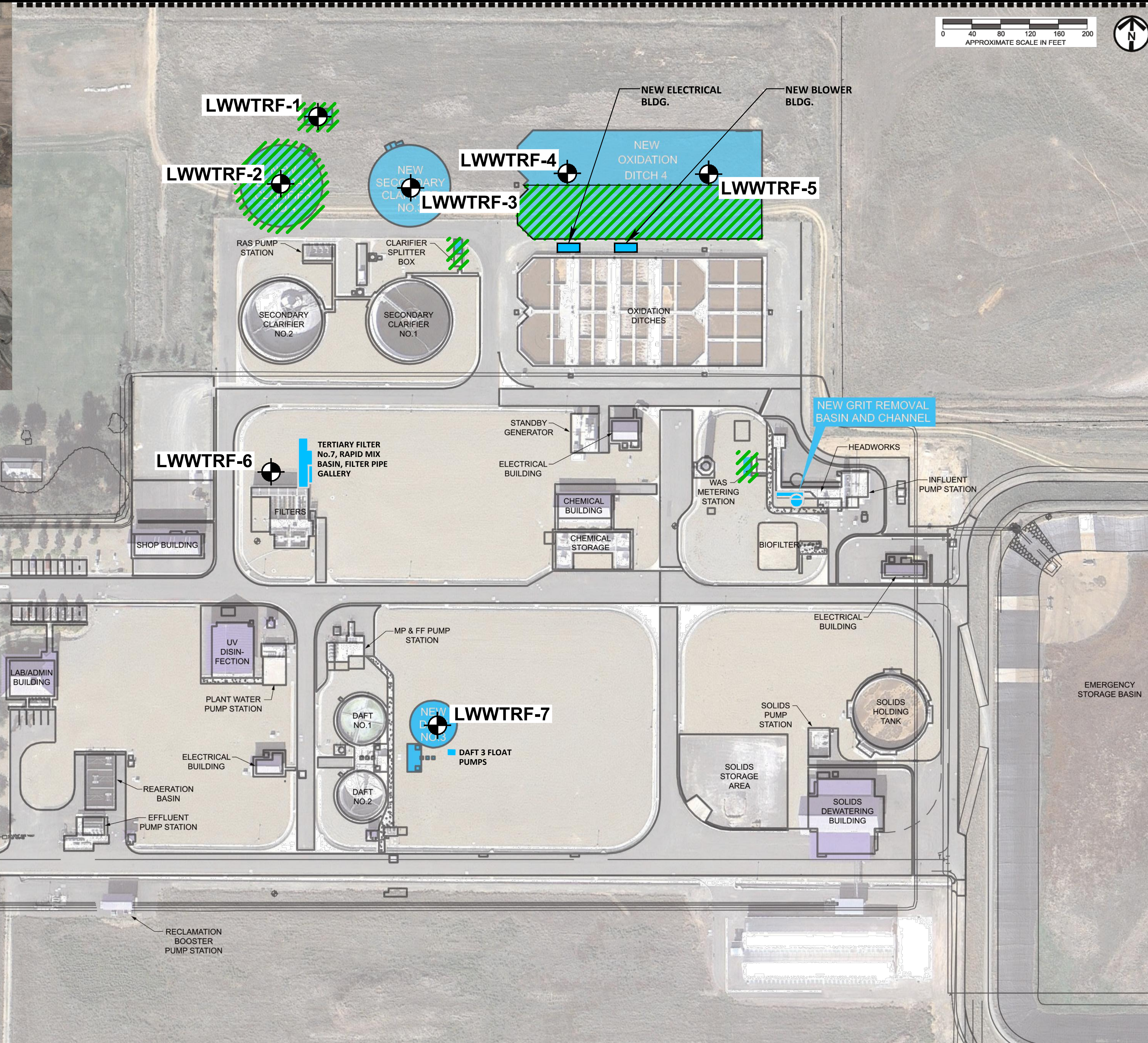
April 2018

Figure 1

4/10/2018 3228.x Fig1 LWWTF Phase 1 and 2 Expansion.dwg



LINCOLN WWTRF



SIGNIFICANT PLANNED STRUCTURAL IMPROVEMENTS

- Grit removal basin and channel
- Oxidation ditch
- Secondary Clarifier
- Dissolved air floatation system (DAFS)
- DAFS pump stations
- Tertiary filter cell

LEGEND

LWWTRF-1 ⊕ Approximate Boring Location (2012), BCI

Previously Constructed Improvements

Map Source: Plans by Stantec Consulting Services Inc., dated February 12, 2012.

BAR IS ONE INCH AT FULL SCALE
0 1"
IF NOT ONE INCH ON THIS SHEET SCALE ACCORDINGLY

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File No. 3228.X

Client / Project
CITY OF LINCOLN
WWTRF PHASE 1 AND PHASE 2 EXPANSION PROJECT
Placer County, California

Project No. 184030200

MDR	JTF	RCP	11/28/17
Drawn	Checked	Designed	YY.MM.DD

CITY OF LINCOLN CALIFORNIA

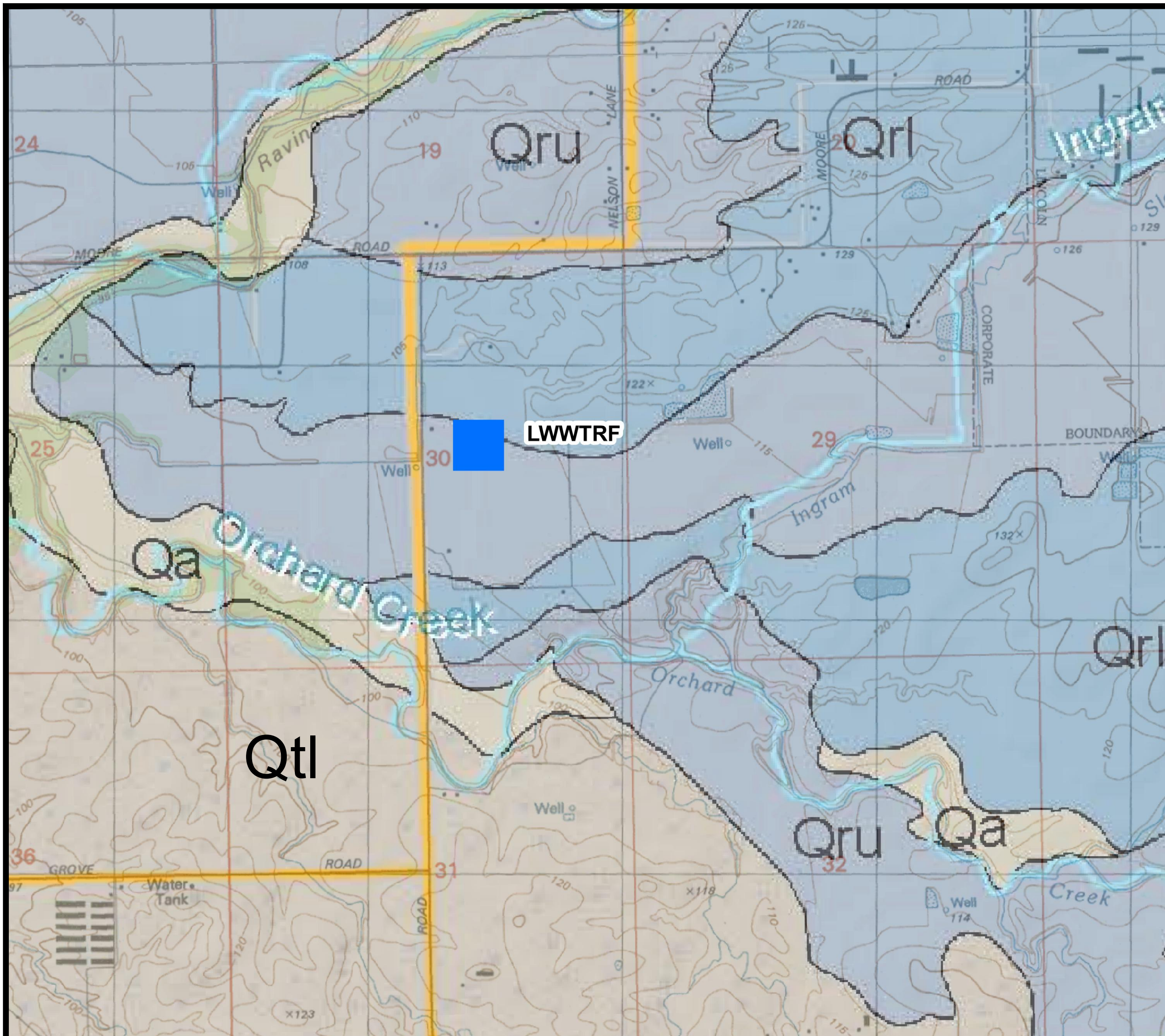
Title
SITE MAP

Figure No.	Sheet	Scale
2	1 of 1	1"=100'

Revision

Y:\Active Projects\3228.x LWWTRF Expansion\CAD\LWWTRF Phase 1 And 2 Figures\3228.x Fig2 LWWTRF Phase 1 And 2 Expansion.dwg 2018/04/10 3:28 PM By: Mike Robertson

ORIGINAL SHEET – ANSI D HORIZ



LEGEND

Qa	Holocene alluvium- silt, sand, and gravel
Qb	Holocene basin deposits- fine grained silt and clay
Qru	Quaternary Upper Member, Riverbank Formation- unconsolidated silt, sand and gravel
Qrl	Quaternary Lower Member, Riverbank Formation- semiconsolidated silt, sand, and gravel
Qtl	Quaternary Turlock Lake Formation- silt, sand, and gravel

N

SCALE 1" = 2,000'

Source: MAPTECH Terrain Navigator Pro, v. 8.0, USGS topographic 7.5 minute quadrangle, Lincoln,1992, Pleasant Grove, 1967 (revised 1981),

Geologic Map of the Late Cenozoic Deposits of the Sacramento Valley and Northern Sierran Foothills, California, Helly, J.H., Hardwood, D.S., USGS, MF-1790, 1985, reproduced by State of California Department of Water Resources, 2006.

4/10/2018 3228.x Fig3 LWWTF Phase 1 and 2 Expansion.dwg

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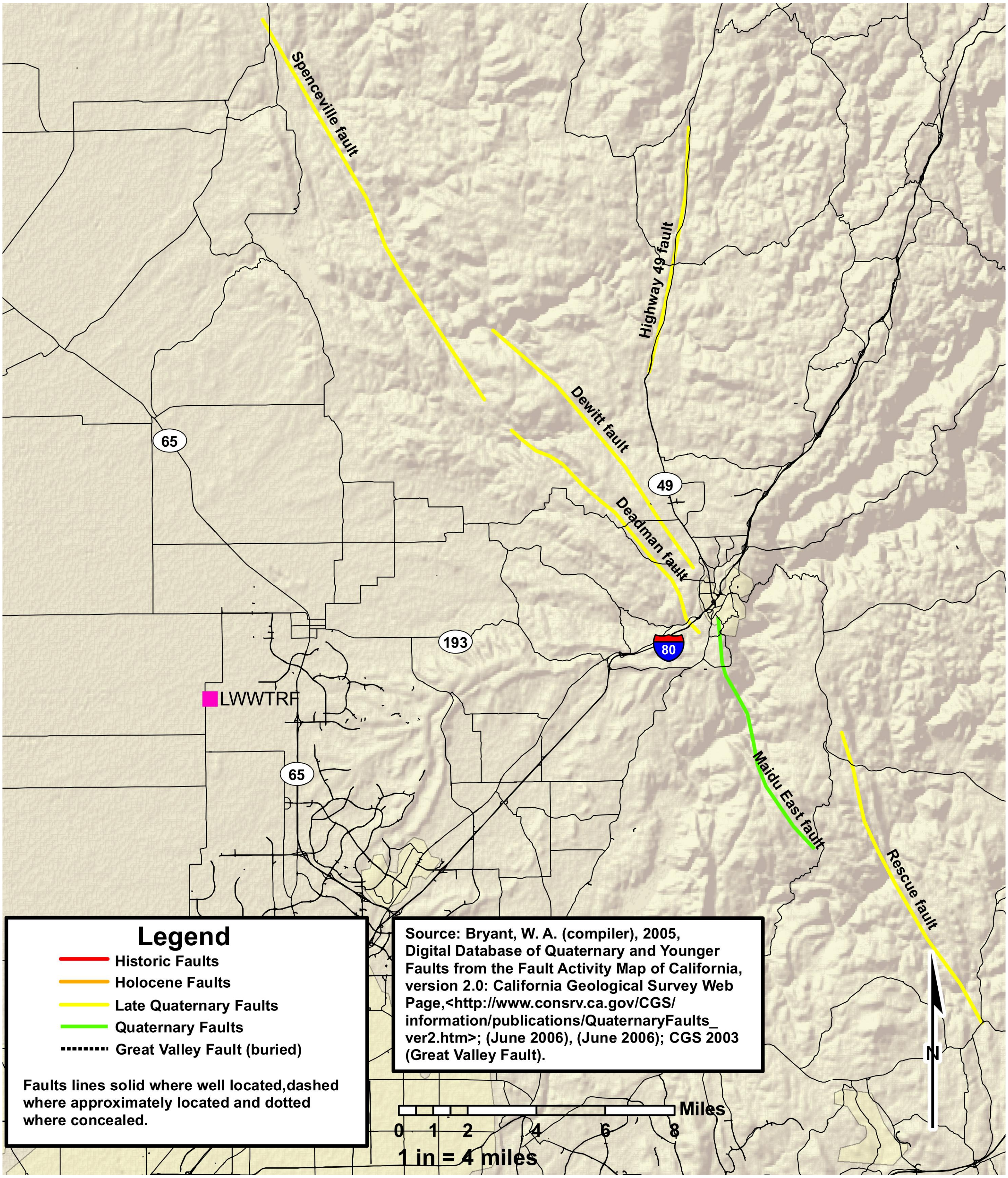
REGIONAL GEOLOGIC MAP

Lincoln Wastewater Treatment and Reclamation Facility, Phase 1 and Phase 2 Expansion Project
Placer County, California

File No. 3228.x

April 2018

Figure 3



4/10/2018 3228.x Fig4 LWWTF Phase 1 and 2 Expansion.dwg

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REGIONAL FAULT MAP

Lincoln Wastewater Treatment and Reclamation Facility, Phase 1 and Phase 2 Expansion Project
Placer County, California

File No. 3228.x

April 2018

Figure 4

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility

Phase 1 and Phase 2 Expansion Project
WWTP Improvements
Placer County, CA

APPENDIX A

Boring Logs (LWWTRF-1 through 7)
Legend of Boring Logs

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RCP	**9-24-12**	**9-24-12**	**38.8633° / -121.34749° NAD83**	**LWWTRF-1**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Taber		**~110.5 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Solid-Stem Auger	**Diedrich D120**	**4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID)	HAMMER TYPE	HAMMER EFFICIENCY, ERi
2.5" Cal Mod	**Safety semi-automatic drop (140#/ 30")**	
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **23.9 ft**; AFTER DRILLING (DATE): **23.9 ft on 9-24-12**	TOTAL DEPTH OF BORING
Boring grout backfilled 9/24/12		**26.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	0	Lean CLAY; CL; hard; olive brown; dry																
108.50	2																	
106.50	4	Lean CLAY; CL; stiff to hard; yellowish brown; moist																
	5		Cal Mod	1	12 36 48	84	100		18	107		18			PP = >4.5	PI		
104.50	6																	
102.50	8																	
100.50	10		Cal Mod	2	8 22 35	57	100								PP = 1.5-2.0			
	10.8	SILTY SAND; SM; very dense; brown; moist; weakly cemented																
98.50	12																	
	13.5	Lean CLAY; CL; hard; yellowish brown; moist																
96.50	14																	
	15																	

(continued)

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PROJECT NAME	FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer	**2110.X**	**LWWTRF-1**
COUNTY: **PLA**	ROUTE	POSTMILE
CLIENT: **Stantec**		
PREPARED BY: **RCP**	CHECKED BY: **PFF**	SHEET: **1 of 2**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	15		Lean CLAY; CL; hard; yellowish brown; moist		3	14	86	100								PP = >4.5			
			SILTY SAND; SM; very dense; brown; moist; weakly cemented			32													
94.50	16					54													
			Lean CLAY; CL; hard; yellowish brown; moist to wet																
	17																		
92.50	18																		
	19																		
90.50	20				4	19	70	100		25	100					PP = >4.5			
						32													
	21					38													
88.50	22																		
	23																		
86.50	24		Groundwater at 23.9 ft																
	25				5	23	77	100											
						36													
84.50	26					41													
	27																		
			Bottom of exploration at 26.5 ft bgs																
82.50	28		Boring grout backfilled 9/24/12																
	29																		
80.50	30																		
	31																		
78.50	32																		
	33																		

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RCP	**9-25-12**	**9-25-12**	**38.86301° / -121.34766° NAD83**	**LWWTRF-2**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Taber		**~110.5 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Solid-Stem Auger	**Diedrich D120**	**4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID)	HAMMER TYPE	HAMMER EFFICIENCY, ERi
2.5" Cal Mod	**Safety semi-automatic drop (140#/ 30")**	
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **22.3 ft**; AFTER DRILLING (DATE): **22.3 ft on 9-25-12**	TOTAL DEPTH OF BORING
Boring grout backfilled 9/25/12		**41.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION/REMARKS	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests
	0	Lean CLAY with SAND; CL; very stiff; light olive brown to brown; dry to moist	Bag B			100									CR
108.50	2														
	3.5	SANDY Lean CLAY; CL; hard; brown; moist													
106.50	4														
	5		1	7 / 9 / 12	21	100		10	113					PP = >4.5	
104.50	6														
	6.5	SILTY SAND; SM; medium dense; brown; moist													
102.50	8														
	8.5	SANDY Lean CLAY; CL; hard; brown; moist													
100.50	10		2	10 / 16 / 17	33	100									
	10.75	SILTY SAND; SM; dense; brown; moist													
	11.4	CLAYEY SAND; SC; dense; brown to greenish gray; moist													
98.50	12														
	13.5	Lean CLAY with SAND; CL; hard; brown; moist													
96.50	14														
	15														

(continued)

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PROJECT NAME		FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer		**2110.X**	**LWWTRF-2**
COUNTY: **PLA**	ROUTE	POSTMILE	
CLIENT: **Stantec**			
PREPARED BY: **RCP**	CHECKED BY: **PFF**		SHEET: **1 of 3**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	15		Lean CLAY with SAND; CL; hard; brown; moist		3	13 30 35	65	100		16	109					PP = >4.5			
94.50	16		SILTY SAND; SM; very dense; brown; moist																
	17		Lean CLAY; CL; hard; light olive brown; moist																
92.50	18																		
	19																		
90.50	20				4	15 48 50/3"	98/9	100		44	76					PP = >4.5 UC =1.93	UC		
	21																		
88.50	22		Groundwater at 22.3 ft																
	23		Lean CLAY; CL; hard; brown; moist																
86.50	24																		
	25				5	11 26 50	76	100		11	101					PP = >4.5			
84.50	26																		
	27																		
82.50	28		Lean CLAY; CL; stiff; brown; wet																
	29																		
80.50	30				6	4 6 8	14	100				87				PP = 1.25-1.75	PA		
	31																		
78.50	32																		
	33																		

(continued)

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PROJECT NAME		FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer		2110.X	LWWTRF-2
COUNTY: PLA	ROUTE	POSTMILE	
CLIENT: Stantec			
PREPARED BY: RCP	CHECKED BY: PFF		SHEET: 2 of 3

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	33		Lean CLAY; CL; stiff; brown; wet																
76.50	34		Lean CLAY; CL; hard; light olive brown; moist																
	35				7	6 13 31	44	100								PP = 4.0->4.5			
74.50	36																		
	37																		
72.50	38		Lean CLAY with SAND; CL; very stiff to hard; mottled light olive brown and brown; moist																
	39																		
70.50	40				8	12 18 30	48	100											
	41		SILTY SAND; SM; dense; brown; moist; weakly cemented																
68.50	42																		
	43		Bottom of exploration at 41.5 ft bgs Boring grout backfilled 9/25/12																
66.50	44																		
	45																		
64.50	46																		
	47																		
62.50	48																		
	49																		
60.50	50																		
	51																		

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PROJECT NAME		FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer		2110.X	LWWTRF-2
COUNTY: PLA	ROUTE	POSTMILE	
CLIENT: Stantec			
PREPARED BY: RCP	CHECKED BY: PFF		SHEET: 3 of 3

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RCP	9-24-12	9-24-12	38.86299° / -121.34708° NAD83	LWWTRF-3

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Taber		~110.5 ft
DRILLING METHOD	**DRILL RIG**	**BOREHOLE DIAMETER**
Solid-Stem Auger	Diedrich D120	4 in
SAMPLER TYPE(S) AND SIZE(S) (ID)	**HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
2.5" Cal Mod	Safety semi-automatic drop (140#/ 30")	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Boring grout backfilled 9/24/12		26.5 ft	26.5 ft on 9-24-12	51.3 ft

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	0	Lean CLAY; CL; stiff to hard; brown to yellowish brown; dry to moist																
	1																	
108.50	2																	
	3																	
106.50	4																	
	5		Cal Mod	1	7 9 15	24	100				61	12				PA, PI		
104.50	6	Sandy Lean CLAY; CL; very stiff; brown; moist																
	7																	
102.50	8																	
	9																	
100.50	10		Cal Mod	2	14 17 19	36	100		24	102					PP = 4.0->4.5			
	11	Lean CLAY; CL; hard; yellowish brown; moist																
98.50	12																	
	13																	
96.50	14																	
	15																	

(continued)

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PROJECT NAME	FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer	2110.X	LWWTRF-3
COUNTY PLA	**ROUTE**	**POSTMILE**
CLIENT Stantec		
PREPARED BY RCP	**CHECKED BY** PFF	**SHEET** 1 of 4

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION/REMARKS	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests
	15	Lean CLAY; CL; hard; yellowish brown; moist	3	13 32 63/5"	95/11	100		23	100					PP = >4.5 UC =3.43	UC
94.50	16	SILTY SAND; SM; very dense; yellowish brown; moist; weakly cemented													
	17														
92.50	18	Lean CLAY; CL; hard; yellowish brown; moist													
	19														
90.50	20		4	29 50 50/4"	100/10	100		30	95					PP = >4.5	
	21														
88.50	22														
	23	Lean CLAY; CL; hard; brown; moist													
86.50	24														
	25		5	21 39 50/5"	89/11	100		21	107					PP = >4.5 UC =4.46	UC
84.50	26	Groundwater at 26.5 ft Lean CLAY; CL; hard; brown to yellowish brown; moist													
	27														
82.50	28	SILT with SAND; ML; stiff; brown; wet													
	29														
80.50	30		6	6 9 11	20	100				96					PA
	31														
78.50	32														
	33														

(continued)

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PROJECT NAME		FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer		2110.X	LWWTRF-3
COUNTY: PLA	ROUTE	POSTMILE	
CLIENT: Stantec			
PREPARED BY: RCP	CHECKED BY: PFF		SHEET: 2 of 4

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	33		SILT with SAND; ML; stiff; brown; wet																
76.50	34		Lean CLAY; CL; hard; yellowish brown; moist																
	35				7	15 35 58	93	100		25	102					PP = >4.5			
74.50	36																		
	37																		
72.50	38		SILTY SAND; SM; dense; brown; wet; weakly cemented																
	39																		
70.50	40				8	15 19 20	39	100											
	41																		
68.50	42																		
	43																		
66.50	44																		
	45				9	16 22 30	52	100		24	104					PP = >4.5			
64.50	46																		
	47																		
62.50	48																		
	49																		
60.50	50				10		93/10	100								PP = >4.5			
	51																		

(continued)

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PROJECT NAME		FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer		2110.X	LWWTRF-3
COUNTY: PLA	ROUTE	POSTMILE	
CLIENT: Stantec			
PREPARED BY: RCP	CHECKED BY: PFF		SHEET: 3 of 4

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	51				10	24 43 50/4"	93/10	100											
58.50	52		Bottom of exploration at 51.3 ft bgs Boring grout backfilled 9/24/12																
	53																		
56.50	54																		
	55																		
54.50	56																		
	57																		
52.50	58																		
	59																		
50.50	60																		
	61																		
48.50	62																		
	63																		
46.50	64																		
	65																		
44.50	66																		
	67																		
42.50	68																		
	69																		

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PROJECT NAME		FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer		2110.X	LWWTRF-3
COUNTY PLA	ROUTE	POSTMILE	
CLIENT Stantec			
PREPARED BY RCP	CHECKED BY PFF		SHEET 4 of 4

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RCP	**9-24-12**	**9-24-12**	**38.86304° / -121.34632° NAD83**	**LWWTRF-4**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Taber		**~110.5 ft**

DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Solid-Stem Auger	**Diedrich D120**	**4 in**

SAMPLER TYPE(S) AND SIZE(S) (ID)	HAMMER TYPE	HAMMER EFFICIENCY, ERi
2.5" Cal Mod	**Safety semi-automatic drop (140#/ 30")**	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Boring grout backfilled 9/24/12		**28.0 ft**	**28.0 ft on 9-24-12**	**31.3 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	0	CLAYEY SAND; SC; dense; brown; dry to moist; weakly cemented																
	1																	
108.50	2																	
	3																	
106.50	4																	
	5			1	5 7 27	34	100								PP = 4.5			
104.50	6																	
	7																	
102.50	8																	
	9																	
100.50	10			2	10 31 63	94	100		12	126					PP = >4.5			
	11																	
98.50	12																	
	13	Lean CLAY; CL; stiff; dark olive brown; moist																
96.50	14																	
	15																	

(continued)

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PROJECT NAME	FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer	**2110.X**	**LWWTRF-4**

COUNTY	ROUTE	POSTMILE
PLA		

CLIENT
Stantec

PREPARED BY	CHECKED BY	SHEET
RCP	**PFF**	

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	15		Lean CLAY; CL; stiff; dark olive brown; moist		3	12 23 43	66	100											
94.50	16		SILTY SAND; SM; very dense; yellowish brown; moist																
	17																		
92.50	18																		
	19		SILTY SAND; SM; very dense; light olive brown; moist; weakly cemented																
90.50	20				4	20 49 50/4"	99/10	100		12	120					PP = >4.5			
	21																		
88.50	22																		
	23		Lean CLAY; CL; hard; mottled brown and light olive gray; moist																
86.50	24																		
	25				5	21 43 50/3"	93/9	100					10			PP = >4.5	PI		
84.50	26																		
	27																		
82.50	28		Groundwater at 28 ft																
	29																		
80.50	30				6	22 45 58/4"	103/10	100								PP = >4.5			
	31																		
78.50	32		Bottom of exploration at 31.3 ft bgs																
	33		Boring grout backfilled 9/24/12																

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PROJECT NAME	FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer	2110.X	LWWTRF-4

COUNTY	ROUTE	POSTMILE
PLA		

CLIENT: Stantec

PREPARED BY	CHECKED BY	SHEET
RCP	PFF	2 of 2

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RCP	9-25-12	9-25-12	38.86305° / -121.3457° NAD83	**LWWTRF-5**

DRILLING CONTRACTOR **Taber**	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION **~110.5 ft**
DRILLING METHOD **Solid-Stem Auger**	DRILL RIG **Diedrich D120**	BOREHOLE DIAMETER **4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **2.5" Cal Mod**	HAMMER TYPE **Safety semi-automatic drop (140#/ 30")**	HAMMER EFFICIENCY, ERi
BOREHOLE BACKFILL AND COMPLETION **Boring grout backfilled 9/25/12**	GROUNDWATER READINGS — DURING DRILLING **27.1 ft** — AFTER DRILLING (DATE) **27.1 ft on 9-25-12**	TOTAL DEPTH OF BORING **41.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	0		Lean CLAY with SAND; CL; stiff to very stiff; light olive brown; dry to moist		Bag A			100									CP		
	1																		
108.50	2																		
	3																		
106.50	4		CLAYEY SAND; SC; dense; olive brown; moist																
	5				1	4 12 19	31	100		16	117					PP = >4.5			
104.50	6																		
	7																		
102.50	8		Lean CLAY; CL; hard; light olive brown; moist																
	9																		
100.50	10				2	18 52/6"	52/6	100		21	95					PP = >4.5 UC =2.55	UC		
	11		SILTY SAND; SM; very dense; brown; moist; weakly cemented																
98.50	12																		
	13																		
96.50	14		Lean CLAY with SAND; CL; hard; light olive brown; moist																
	15																		

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PROJECT NAME	FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer	2110.X	LWWTRF-5

COUNTY	ROUTE	POSTMILE
PLA		

CLIENT		
Stantec		

PREPARED BY	CHECKED BY	SHEET
RCP	PFF	1 of 3

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	15		Lean CLAY with SAND; CL; hard; light olive brown; moist		3	25 50/5"	50/5	100								PP = >4.5			
94.50	16																		
	17																		
92.50	18		Lean CLAY; CL; hard; dark brown to light olive brown; moist																
	19																		
90.50	20				4	12 21 31	52	100		24	101					PP = >4.5			
	21																		
88.50	22																		
	23																		
86.50	24																		
	25				5	15 32 50	82	100		32	91					PP = >4.5	CR		
84.50	26																		
	27		Groundwater at 27.1 ft																
82.50	28																		
	29		Lean CLAY with SAND; CL; very stiff; light olive brown; moist																
80.50	30				6	12 20 54	74	100								PP = 3.5-3.75			
	31																		
78.50	32																		
	33																		

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PROJECT NAME	FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer	2110.X	LWWTRF-5

COUNTY	ROUTE	POSTMILE
PLA		

CLIENT: Stantec

PREPARED BY	CHECKED BY	SHEET
RCP	PFF	2 of 3

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	33		Lean CLAY; CL; hard; brown; moist																
76.50	34																		
	35				7	12 20 33	53	100		21	110					PP = >4.5			
74.50	36																		
	37																		
72.50	38		SANDY Lean CLAY; CL; hard; brown; moist																
	39																		
70.50	40				8	17 27 47	74	100								PP = >4.5			
	41																		
68.50	42		Bottom of exploration at 41.5 ft bgs																
	43		Boring grout backfilled 9/25/12																
66.50	44																		
	45																		
64.50	46																		
	47																		
62.50	48																		
	49																		
60.50	50																		
	51																		

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PROJECT NAME	FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer	2110.X	LWWTRF-5

COUNTY	ROUTE	POSTMILE
PLA		

CLIENT: Stantec

PREPARED BY	CHECKED BY	SHEET
RCP	PFF	3 of 3

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RCP	**9-25-12**	**9-25-12**	**38.86191° / -121.34771° NAD83**	**LWWTRF-6**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Taber		**~110.5 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Solid-Stem Auger	**Diedrich D120**	**4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID)	HAMMER TYPE	HAMMER EFFICIENCY, ERi
2.5" Cal Mod	**Safety semi-automatic drop (140#/ 30")**	
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **None**; AFTER DRILLING (DATE): **None**	TOTAL DEPTH OF BORING
Boring grout backfilled 9/25/12		**21.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	0	Lean CLAY; CL; stiff to very stiff; brown; moist																
	1																	
108.50	2																	
	3																	
106.50	4	Lean CLAY with SAND; CL; hard; medium gray to brown; moist																
	5		Cal Mod	1	4 10 15	25	100		17	118					PP = >4.5			
104.50	6																	
	7																	
102.50	8																	
	9																	
100.50	10		Cal Mod	2	9 18 17	35	100											
	11	SILTY SAND; SM; dense; brown; moist																
98.50	12																	
	13																	
96.50	14																	
	15																	

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PROJECT NAME	FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer	**2110.X**	**LWWTRF-6**
COUNTY: **PLA**	ROUTE	POSTMILE
CLIENT: **Stantec**		
PREPARED BY: **RCP**	CHECKED BY: **PFF**	SHEET: **1 of 2**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	15		Lean CLAY with SAND; CL; very stiff; brown; moist		3	13	50	100		15	114					PP = 3.75			
			SILTY SAND; SM; very dense; brown; moist; weakly cemented			22													
94.50	16					28													
	17																		
92.50	18		Lean CLAY; CL; very stiff; light olive brown; moist																
	19																		
90.50	20				4	8	26	100								PP = 3.75-4.0			
						11													
	21					15													
88.50	22		Bottom of exploration at 21.5 ft bgs																
	23		Boring grout backfilled 9/25/12																
86.50	24																		
	25																		
84.50	26																		
	27																		
82.50	28																		
	29																		
80.50	30																		
	31																		
78.50	32																		
	33																		

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PROJECT NAME		FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer		2110.X	LWWTRF-6
COUNTY: PLA	ROUTE	POSTMILE	
CLIENT: Stantec			
PREPARED BY: RCP	CHECKED BY: PFF		SHEET: 2 of 2

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RCP	**9-25-12**	**9-25-12**	**38.86093° / -121.34693° NAD83**	**LWWTRF-7**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Taber		**~110.5 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Solid-Stem Auger	**Diedrich D120**	**4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID)	HAMMER TYPE	HAMMER EFFICIENCY, ERi
2.5" Cal Mod	**Safety semi-automatic drop (140#/ 30")**	
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **22.9 ft**; AFTER DRILLING (DATE): **22.9 ft on 9-25-12**	TOTAL DEPTH OF BORING
Boring grout backfilled 9/25/12		**36.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	0	Lean CLAY; CL; hard; brown to dark brown; moist																
	1																	
108.50	2																	
	3																	
106.50	4																	
	5			1	4 9 15	24	100		16	117		16			PP = >4.5	PI		
104.50	6																	
	7																	
102.50	8																	
	9																	
100.50	10			2	6 12 14	26	100											
	11	SILTY SAND; SM; medium dense; brown; moist																
98.50	12																	
	13																	
96.50	14	Lean CLAY; CL; hard; light olive brown; moist to wet																
	15																	

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PROJECT NAME		FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer		**2110.X**	**LWWTRF-7**
COUNTY **PLA**	ROUTE	POSTMILE	
CLIENT **Stantec**			
PREPARED BY **RCP**	CHECKED BY **PFF**		SHEET **1 of 3**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION/REMARKS	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests
	15	Lean CLAY; CL; hard; light olive brown; moist to wet	3	7 10 14	24	100		20	108					PP = >4.5 UC =2.6	CR, UC
94.50	16														
	17														
92.50	18														
	19														
90.50	20		4	10 22 28	50	100		18	113					PP = >4.5	
	21														
88.50	22														
	23	Groundwater at 22.9 ft													
86.50	24														
	25		5	2 4 6	10	0									
84.50	26														
	27														
82.50	28	Lean CLAY with SAND; CL; very stiff; light olive brown; moist													
	29														
80.50	30		6	9 11 12	23	100								PP = 3.25-3.5	
	31														
78.50	32														
	33														

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PROJECT NAME	FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer	2110.X	LWWTRF-7

COUNTY	ROUTE	POSTMILE
PLA		

CLIENT: Stantec

PREPARED BY	CHECKED BY	SHEET
RCP	PFF	2 of 3

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Type	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	33		Lean CLAY with SAND; CL; very stiff; light olive brown; moist																
76.50	34																		
	35				7	14 22 25	47	100											
74.50	36																		
	37		Bottom of exploration at 36.5 ft bgs Boring grout backfilled 9/25/12																
72.50	38																		
	39																		
70.50	40																		
	41																		
68.50	42																		
	43																		
66.50	44																		
	45																		
64.50	46																		
	47																		
62.50	48																		
	49																		
60.50	50																		
	51																		

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PROJECT NAME		FILE NO.	HOLE ID
Mid-Western Placer Regional Sewer		2110.X	LWWTRF-7
COUNTY: PLA	ROUTE	POSTMILE	
CLIENT: Stantec			
PREPARED BY: RCP	CHECKED BY: PFF		SHEET: 3 of 3

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (CTM 216 - 06)
CR	Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content (ASTM D 2974-07)
P	Permeability (CTM 220 - 05)
PA	Particle Size Analysis (ASTM D 422-63 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301 - 00)
SE	Sand Equivalent (CTM 217 - 99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96 [2004])

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- 2.5" ID Sampler
- 2" ID Sampler
- Shelby Tube
- Piston Sampler
- NX Rock Core
- HQ Rock Core
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (short-term)
- Static Water Level Reading (long-term)

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BORING RECORD LEGEND

COUNTY	ROUTE	POSTMILE	
Placer			
PROJECT NAME			
Mid-Western Placer Regional Sewer			
File No. 2110.X	PREPARED BY RCP	DATE	SHEET 1 of 3

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60} - Value (blows / foot)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Descriptor	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Descriptor	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE: This legend sheet provides descriptors and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (July 2007), Section 2, for tables of additional soil description components and discussion of soil description and identification.

blackburn consulting

Blackburn Consulting
11521 Blocker Drive, Suite 110
Auburn, CA 95603
Phone: (530) 887-1494
Fax: (530) 887-1495

BORING RECORD LEGEND

COUNTY	ROUTE	POSTMILE	
Placer			
PROJECT NAME			
Mid-Western Placer Regional Sewer			
File No. 2110.X	PREPARED BY RCP	DATE	SHEET 2 of 3

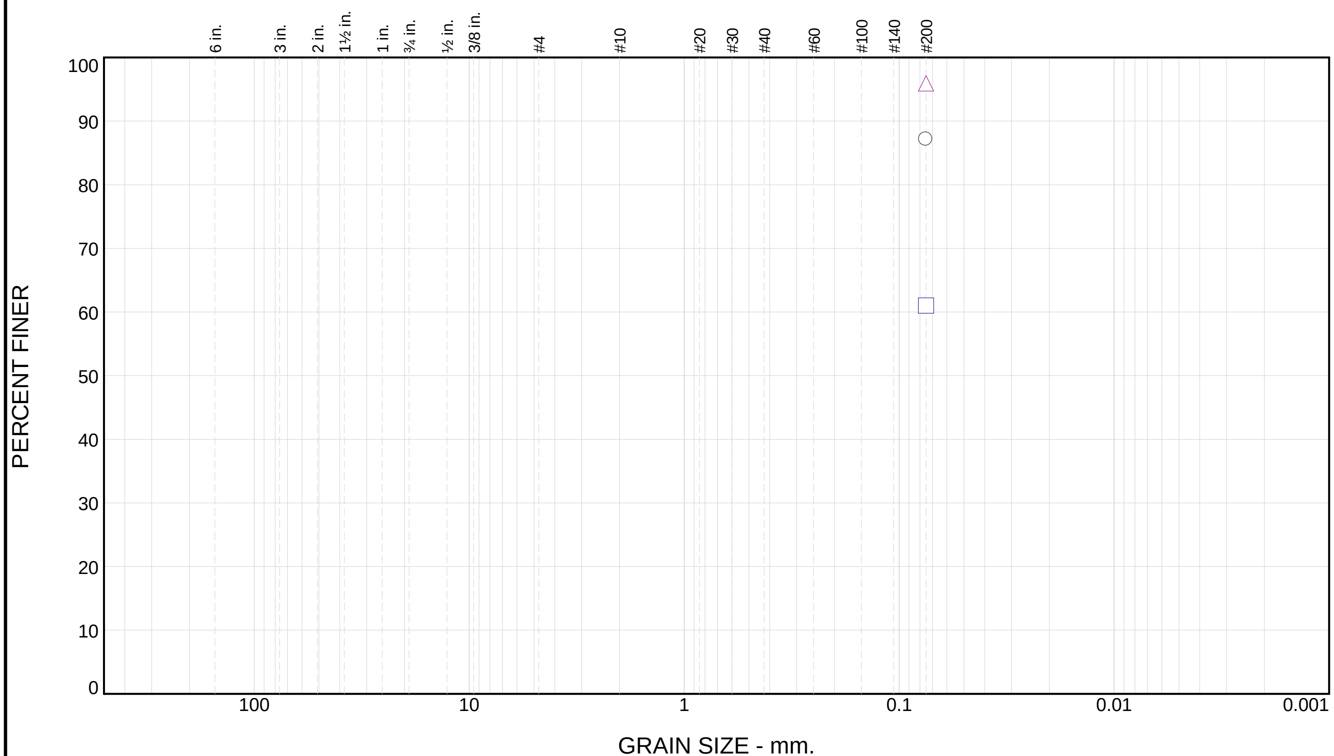
GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
WWTP Improvements
Placer County, CA

APPENDIX B

Laboratory Test Results

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
○						CL			
□						CL	28	16	12
△						ML			

SIEVE inches size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △

	GRAIN SIZE ○	GRAIN SIZE □	GRAIN SIZE △
D_{60}			
D_{30}			
D_{10}			

	COEFFICIENTS ○	COEFFICIENTS □	COEFFICIENTS △
C_c			
C_u			

SIEVE number size	PERCENT FINER ○	PERCENT FINER □	PERCENT FINER △
#200	87.1	61.0	95.9

Material Description

○ Yellowish Brown Lean CLAY

□ Brown Sandy Lean CLAY

△ Yellowish Brown SILT

REMARKS:

○

□

△

○ Depth: 30.5'-31.0' Sample Number: LWWTRF-2-6B

□ Depth: 6.0'-6.5' Sample Number: LWWTRF-3-1C

△ Depth: 30.5'-31.0' Sample Number: LWWTRF-3-6B

Blackburn Consulting

Auburn, CA

Client: Stantec

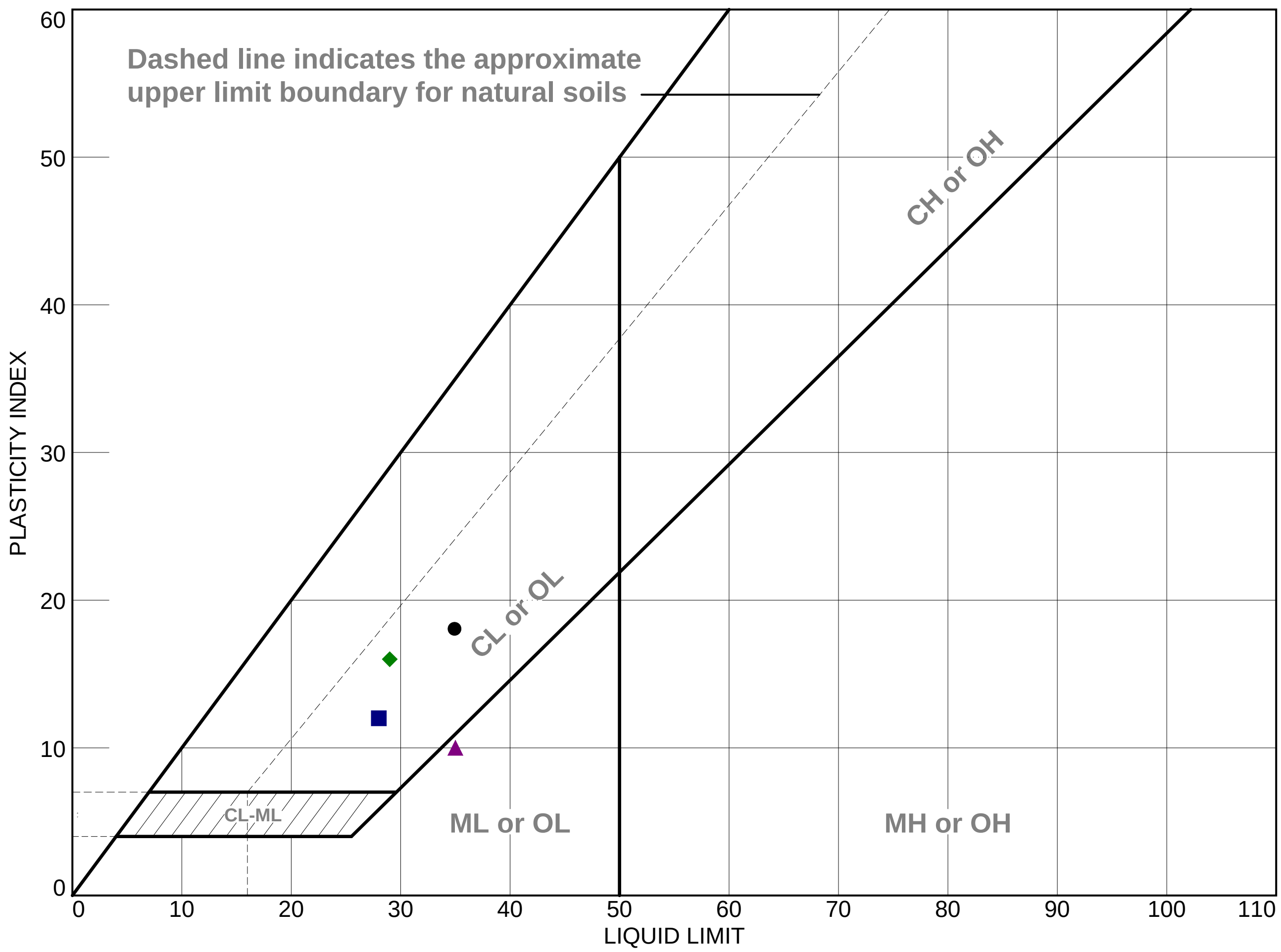
Project: Mid Western Placer Regional Sewer

Project No.: 2110.x

Figure

Tested By: KLC **Checked By:** KLC

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		LWWTRF-1-1B	5.5'-6.0'		17	35	18	
■		LWWTRF-3-1C	6.0'-6.5'		16	28	12	CL
▲		LWWTRF-4-	25.25'-25.75'		25	35	10	

Blackburn Consulting

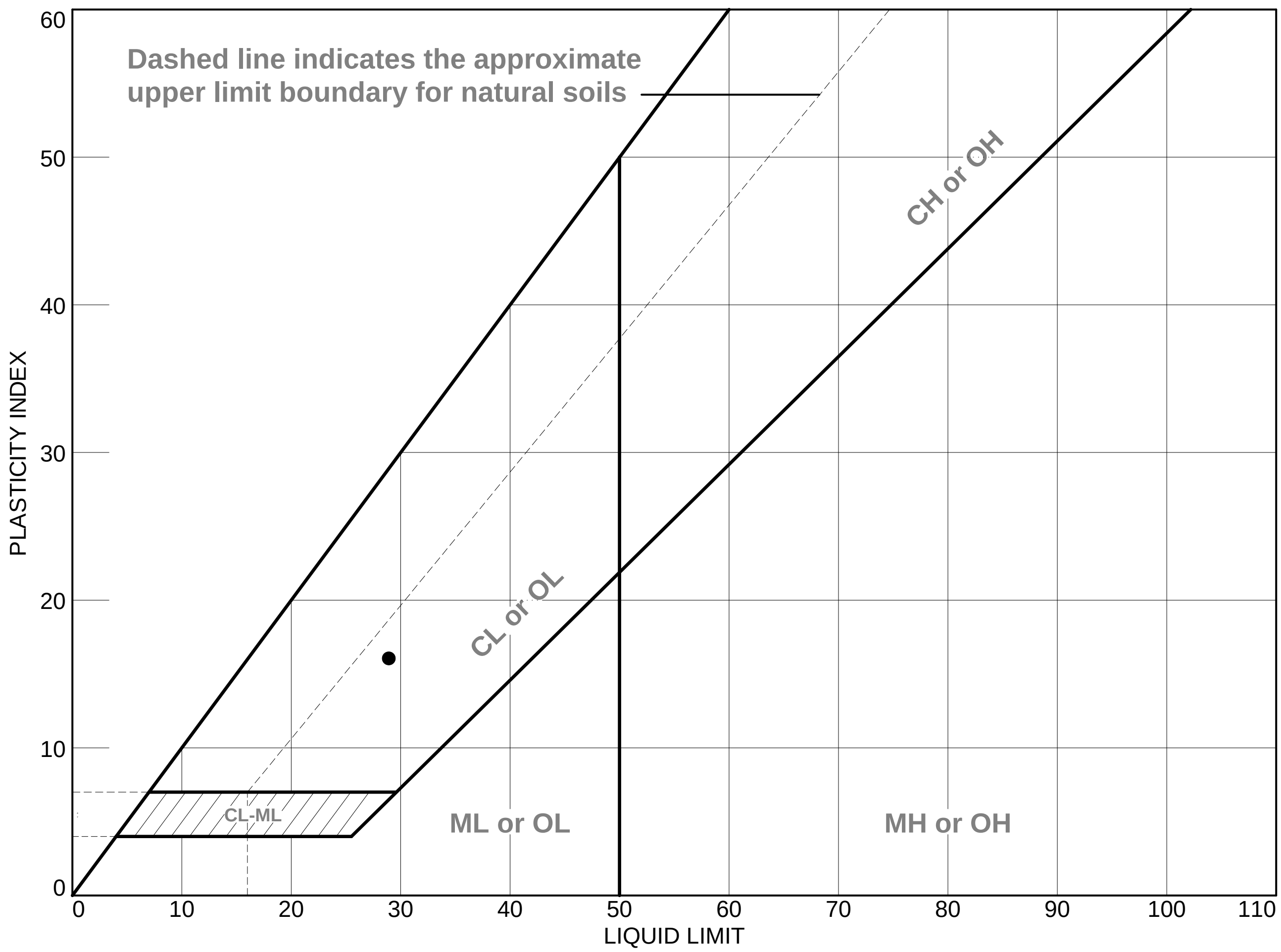
Auburn, CA

Client: Stantec
Project: Mid Western Placer Regional Sewer

Project No.: 2110.x **Figure**

Tested By: ○ KLC □ KLC △ KLC ◇ KlC **Checked By:** RP

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●		LWWTRF-7-1B	5.5'-6.0'		13	29	16	

Blackburn Consulting

Auburn, CA

Client: Stantec
Project: Mid Western Placer Regional Sewer

Project No.: 2110.x **Figure**

Tested By: KIC **Checked By:** RP

DRGSRKRI H 5 SP TU WWSR CI WX
3 BC8 7 . - 00

Project Name: Mid Western Placer Regional Sewer
Project Number: 2110.X
Sample: LWWTRF-B2 #4c Depth: 20.75-21.25'
Sample Description: Lean CLAY, yellowish brown (cemented)
Date: 1/28/2013
Tested By: KAC

blackburn consulting

Original Sample Length	5.97
Original Diameter (in)	2.40
Height-to-Diameter Ratio	2.5 : 1
Sample Area (in^2)	4.52

Test Results

Rate of Strain (in/min)	0.060	(1%/min)
Average cross-sectional area (in^2)	4.62	
Deflection at Max. Load (in)	0.128	
Maximum Load (lbs)	124	
Strain at Failure (%)	2.1	
Compressive Strength (tsf)	- +1/	

Moisture Density

AI P EUNW2

* % moisture taken after test.

Tube and Sample (g)	1061.20
Tube (g)	286.30
Sample Weight (g)	774.90
Tare Number	B7
Tare Weight (g)	152.70
Wet Weight (g)	607.50
Dry Weight (g)	469.10
Dry Weight (g)	316.40
Water Weight (g)	138.40
Percent Moisture (%)*	43.7
Wet Density (pcf)	109.4
Dry Density (pcf)	76.1

Compression Tests

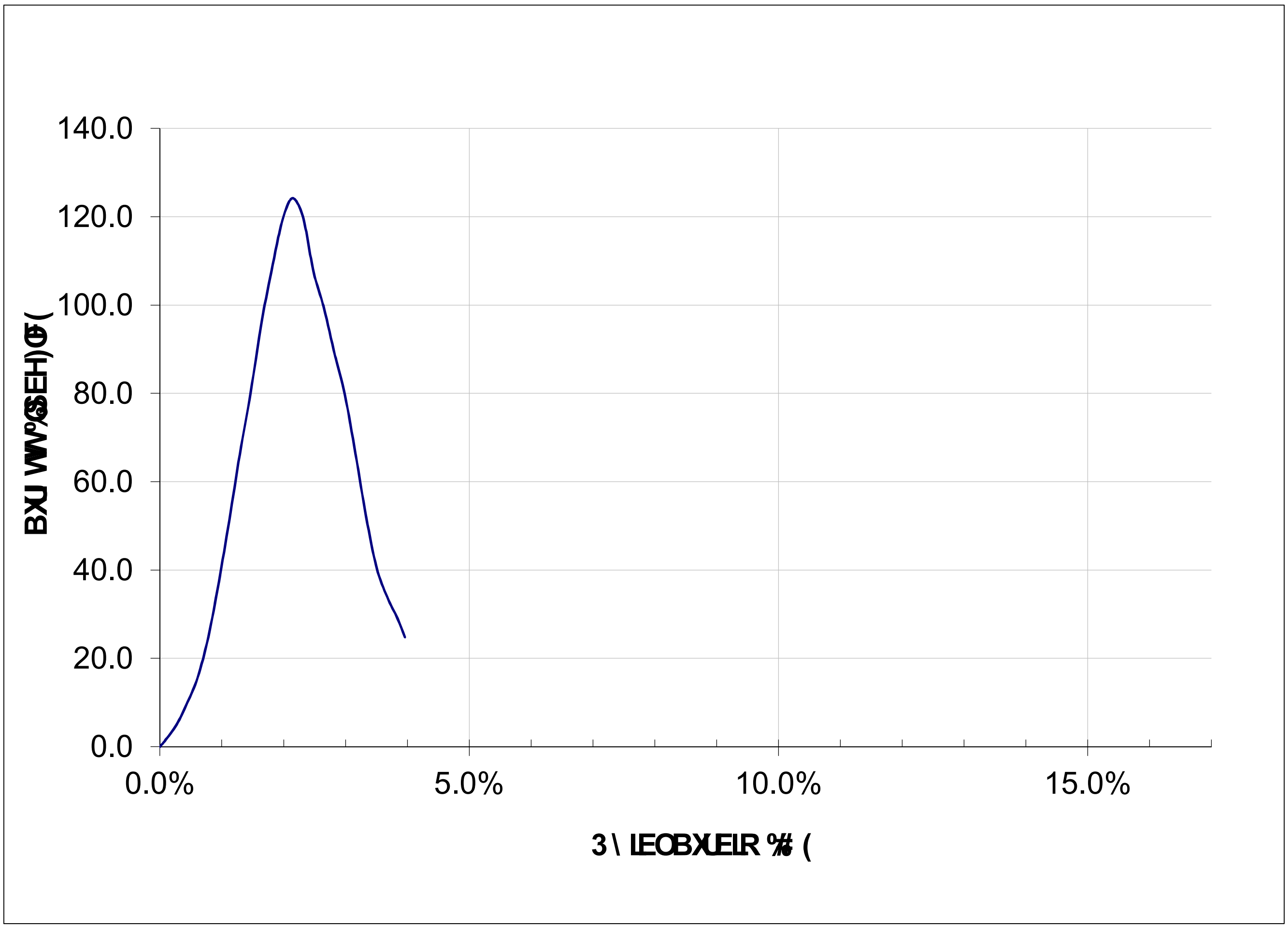
Dial reading @ 0 lb	0.000

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.006	2	0.169	89				
0.016	5	0.179	80				
0.026	10	0.189	66				
0.036	16	0.199	52				
0.047	25	0.209	41				
0.057	37	0.219	34				
0.067	51	0.229	29				
0.077	66	0.236	25				
0.088	80						
0.097	95						
0.108	108						
0.118	119						
0.128	124						
0.138	120						
0.148	108						
0.158	99						

=USMIGX
Mid Western Placer Regional Sewer
=USMIGX: ZP FI U
2110.X
BEP TO : ZP FI U
LWWTRF-B2 #4c
8 EXI UEO7 I WGLITXSR
Lean CLAY, yellowish brown (cemented)
CI WXI H 4 a
KAC

3 BC8 7 . - 00



Wet Density (pcf)	109.4
Dry Density (pcf)	76.1
% Moisture	43.7
Unconfined Compressive Strength (tsf)	1.93

ESHTSLMSKI 7TR UWKXXMTS DKXZ

4CD: 8 / . 22)-2

Project Name: Mid Western Placer Regional Sewer

Project Number: 2110.X

Sample: LWWTRF B3-3c Depth: 15.9-16.4'

Sample Description: SILTY SAND, light olive brown (Partially Cemented)

Date: 10/22/2012

Tested By: B. Moore

Original Sample Length	6.00
Original Diameter (in)	2.40
Height-to-Diameter Ratio	2.5 : 1
Sample Area (in^2)	4.52

Test Results

Axial Strain at Max. Load	3.6%
Average cross-sectional area (in^2)	4.69
Deflection at Max. Load (in)	0.213
Maximum Load (lbs)	223
Strain at Failure (%)	1.28
Compressive Strength (tsf)	**0+10**

Moisture Density

BKR FWOX3

* % moisture taken after test.

Tube and Sample (g)	1141.90
Tube (g)	266.50
Sample Weight (g)	875.40
Tare Number	A7
Tare Weight (g)	153.80
Wet Weight (g)	556.90
Dry Weight (g)	481.90
Dry Weight (g)	328.10
Water Weight (g)	75.00
Percent Moisture (%)*	22.9
Wet Density (pcf)	122.9
Dry Density (pcf)	100.0

Compression Tests

Dial reading @ 0 lb	0.000

Rate of Strain=0.056in/min

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
	2	0.162	154				
0.010	11	0.173	171				
0.021	16	0.183	187				
0.030	20	0.193	201				
0.041	25	0.203	213				
0.051	30	0.213	223				
0.061	36	0.224	222				
0.071	42	0.233	208				
0.081	50	0.244	142				
0.092	59	0.254	42				
0.101	69	0.264	41				
0.112	79	0.274	31				
0.122	93	0.285	29				
0.132	107	0.285	26				
0.142	122						
0.153	138						

AWTNKHZ

Mid Western Placer Regional Sewer

AWTNKHZ=\ R GKW

2110.X

CFR UPK=\ R GKW

LWWTRF B3-3c

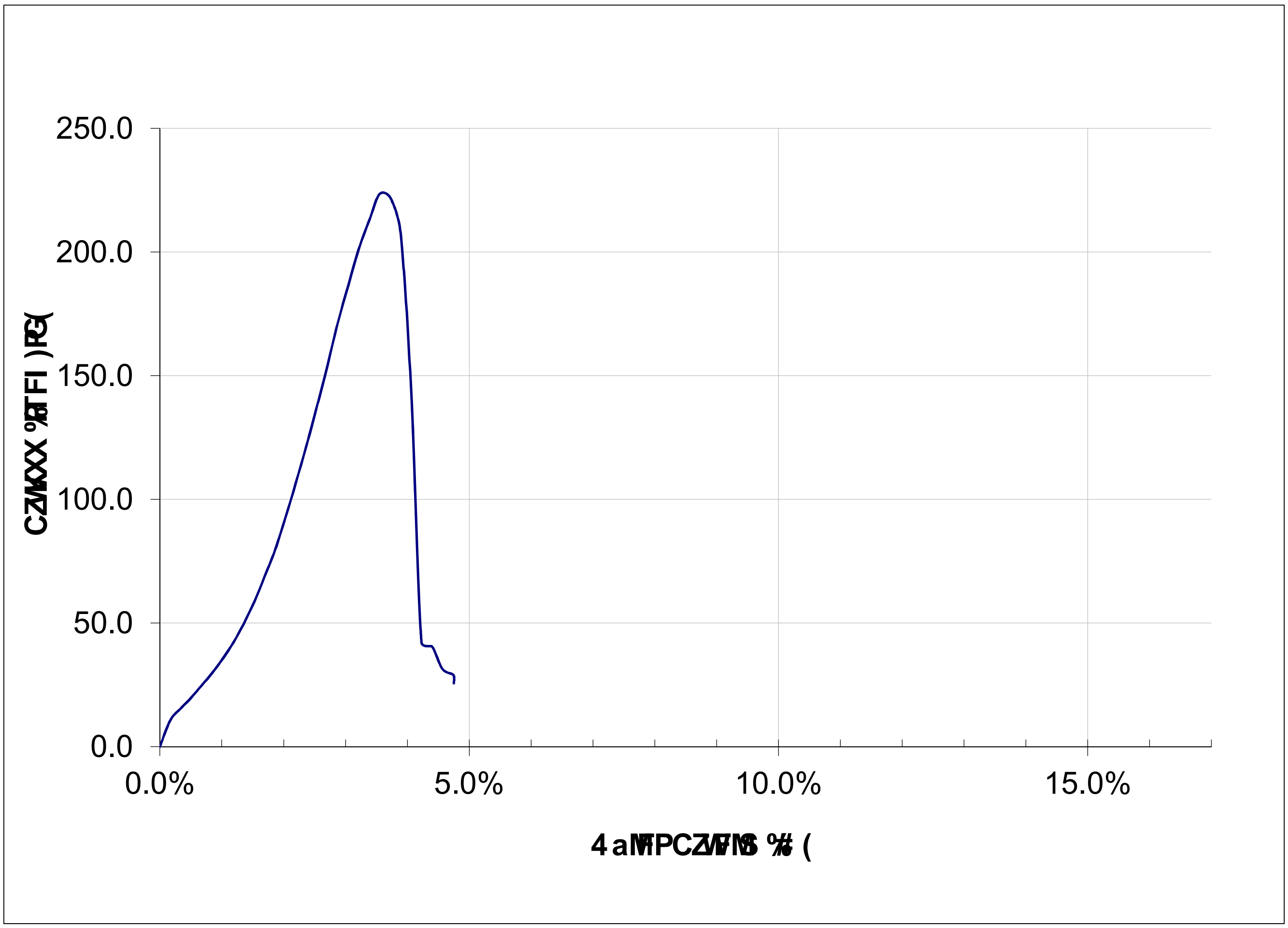
: FZKWMP8 KXHMUZMS

SILTY SAND, light olive brown (Partially Cemented)

DKXZKI 5b

B. Moore

4CD: 8 / . 22)- 2



Wet Density (pcf)	122.9
Dry Density (pcf)	100.0
% Moisture	22.9

Unconfined Compressive Strength (tsf)	3.43

DRGSRKRI H 5 SP TU WWSR CI WX
3 BC8 7 / . 11)- 1

Project Name: Mid Western Placer Regional Sewer
Project Number: 2110.X
Sample: LWWTRF-B3 #5c Depth: 26.0-26.5'
Sample Description: Lean CLAY, yellowish red (cemented)
Date: 1/30/2013
Tested By: KAC

Original Sample Length	5.98
Original Diameter (in)	2.40
Height-to-Diameter Ratio	2.5 : 1
Sample Area (in^2)	4.52

Test Results

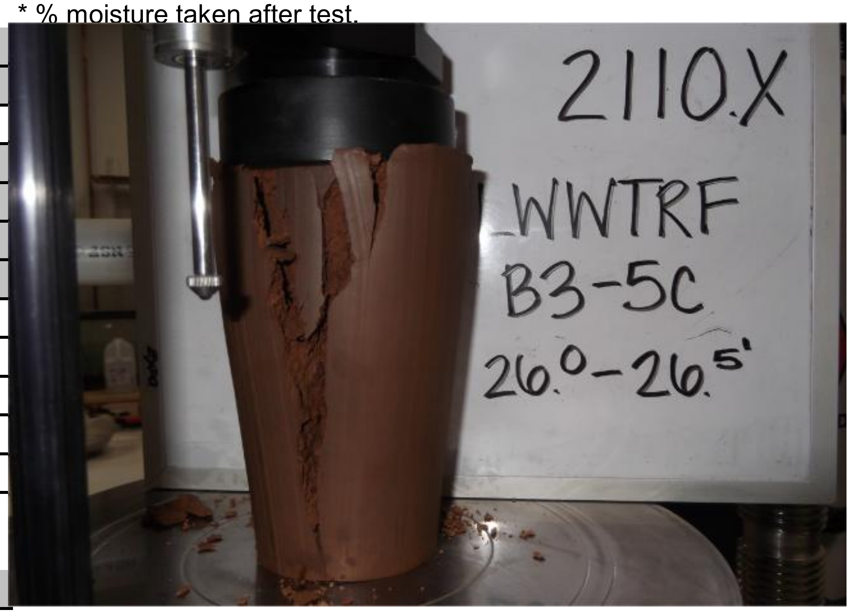
Rate of Strain (in/min)	0.060	(1%/min)
Average cross-sectional area (in^2)	4.69	
Deflection at Max. Load (in)	0.206	
Maximum Load (lbs)	291	
Strain at Failure (%)	3.4	
Compressive Strength (tsf)	**0+01**	

Moisture Density

AI P EUNW2

* % moisture taken after test.

Tube and Sample (g)	1201.70
Tube (g)	286.40
Sample Weight (g)	915.30
Tare Number	B6
Tare Weight (g)	154.10
Wet Weight (g)	588.30
Dry Weight (g)	513.30
Dry Weight (g)	359.20
Water Weight (g)	75.00
Percent Moisture (%)*	20.9
Wet Density (pcf)	128.9
Dry Density (pcf)	106.6



Compression Tests

Dial reading @ 0 lb	0.000

Rate of Strain=0.056in/min

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.022	15	0.328	43				
0.042	30	0.348	45				
0.044	45	0.369	48				
0.064	84	0.389	52				
0.084	131	0.409	56				
0.105	176	0.429	59				
0.125	208	0.450	60				
0.145	236	0.470	63				
0.166	258	0.490	56				
0.186	278						
0.206	291						
0.227	251						
0.247	156						
0.267	91						
0.287	50						
0.308	43						

=USMIGX

Mid Western Placer Regional Sewer

=USMIGX: ZP FI U

2110.X

BEP TO : ZP FI U

LWWTRF-B3 #5c

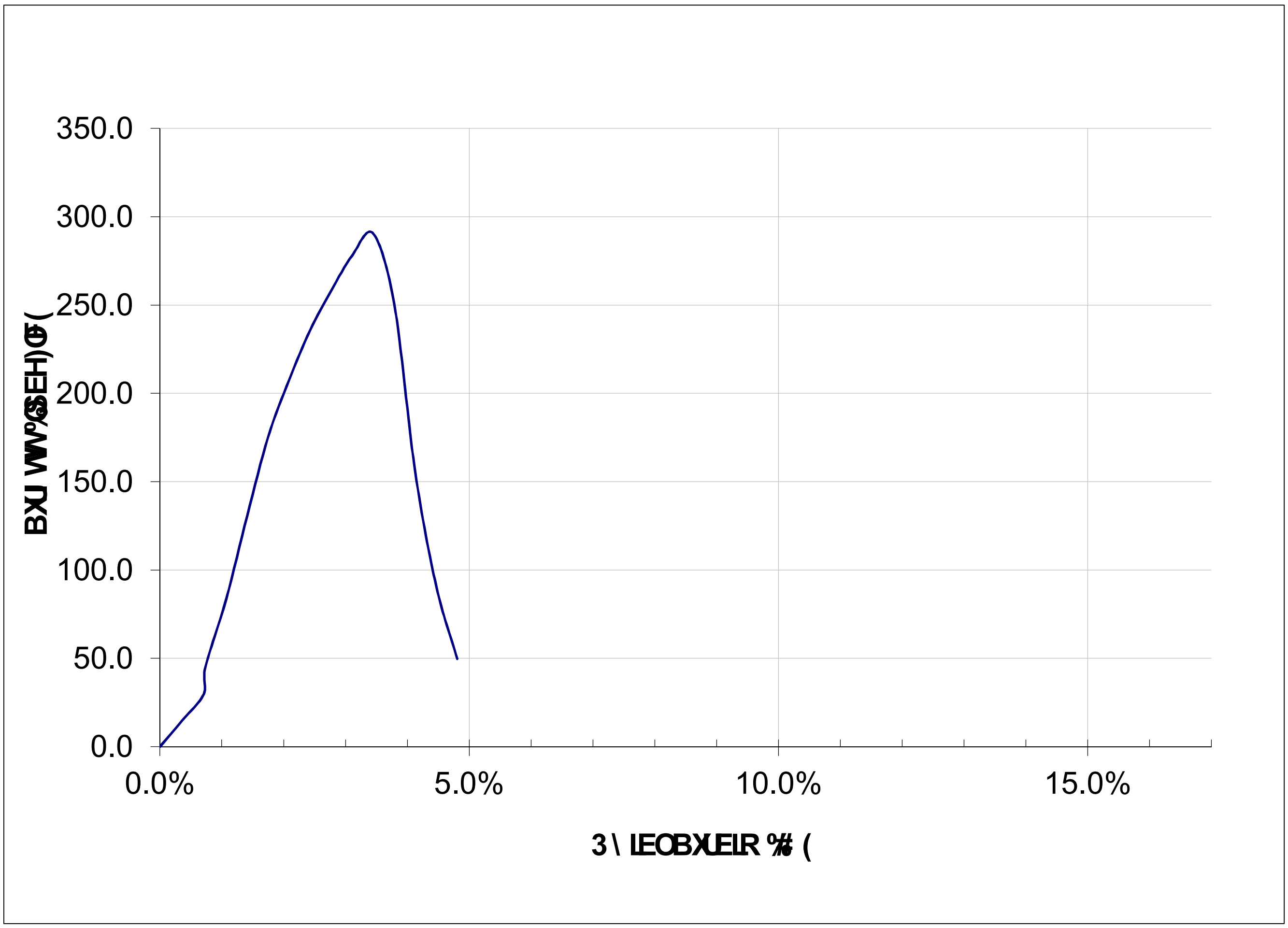
8 EXI UEO7 I WGUTXSR

Lean CLAY, yellowish red (cemented)

CI WXI H 4a

KAC

3BC8 7 / . 11)- 1



Wet Density (pcf)	128.9
Dry Density (pcf)	106.6
% Moisture	20.9

Unconfined Compressive Strength (tsf) 4.46

DRGSRKKRI H 5 SP TU WWSR CI WX

3 BC8 7 / . 11)- 1

Project Name: Mid Western Placer Regional Sewer

Project Number: 2110.X

Sample: LWWTRF-B5 #2c   Depth: 11.0-11.5'

Sample Description: Lean CLAY (top)/SILTY SAND (bottom), yellowish brown (cemented)

Date: 1/30/2013

Tested By: KAC

Original Sample Length	5.99
Original Diameter (in)	2.40
Height-to-Diameter Ratio	2.5 : 1
Sample Area (in^2)	4.52

Test Results

Rate of Strain (in/min)	0.060	(1%/min)
Average cross-sectional area (in^2)	4.60	
Deflection at Max. Load (in)	0.102	
Maximum Load (lbs)	163	
Strain at Failure (%)	1.7	
Compressive Strength (tsf)	/ +00	

Moisture Density

AI P EUNW2

* % moisture taken after test.

Tube and Sample (g)	1023.80
Tube (g)	211.50
Sample Weight (g)	812.30
Tare Number	C1
Tare Weight (g)	153.00
Wet Weight (g)	639.50
Dry Weight (g)	556.90
Dry Weight (g)	403.90
Water Weight (g)	82.60
Percent Moisture (%)*	20.5
Wet Density (pcf)	114.3
Dry Density (pcf)	94.9

Compression Tests

Dial reading @ 0 lb	0.000

Rate of Strain=0.056in/min

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.011	2						
0.021	9						
0.031	22						
0.042	36						
0.052	54						
0.062	76						
0.072	99						
0.082	125						
0.092	149						
0.102	163						
0.113	124						
0.123	10						
0.133	10						
0.143	11						

=USMIGX

Mid Western Placer Regional Sewer

=USMIGX: ZP FI U

2110.X

BEP TO : ZP FI U

LWWTRF-B5 #2c

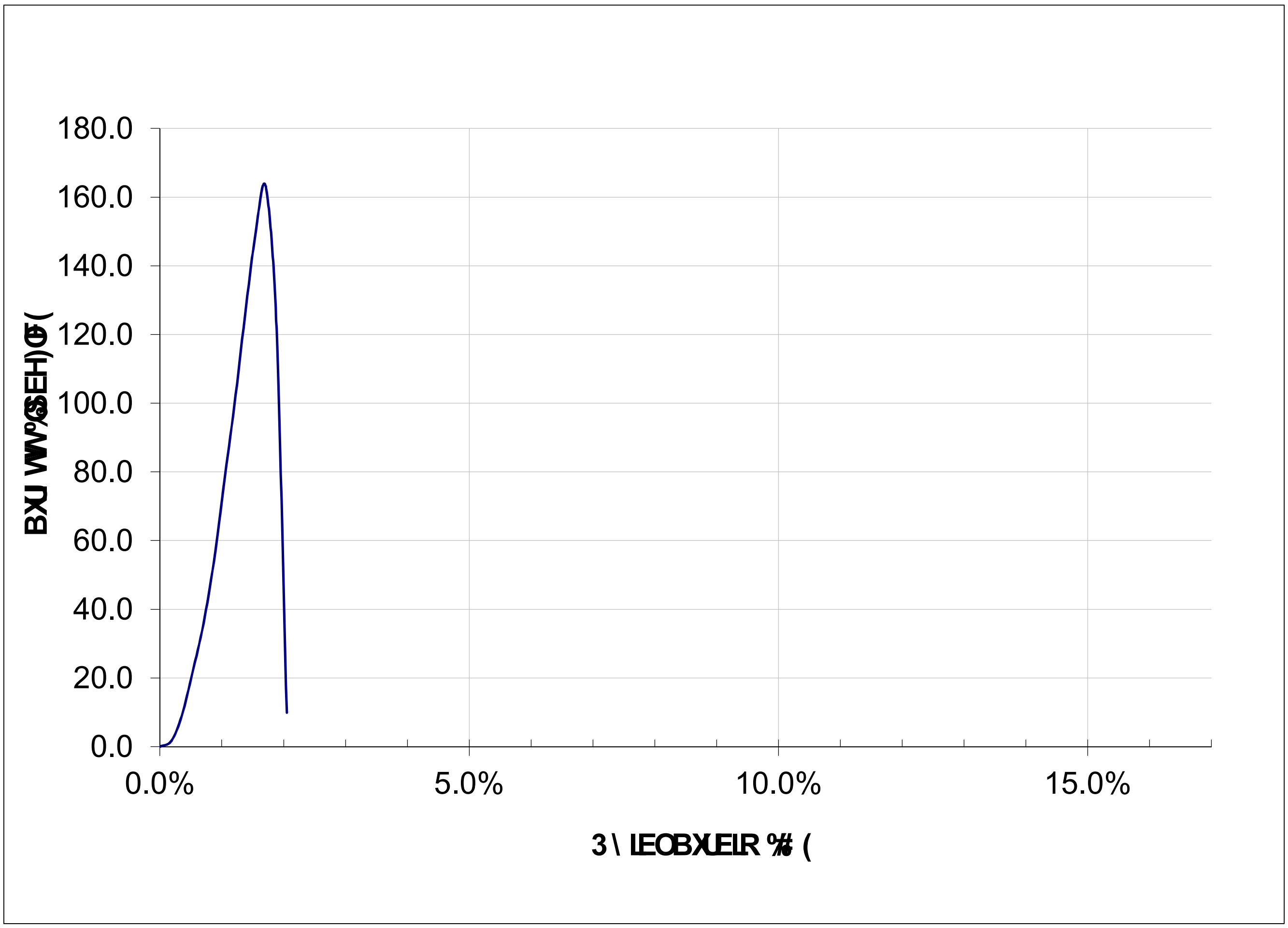
8 EX UEO7 I WGUTXSR

Lean CLAY (top)/SILTY SAND (bottom), yellowish brown (cemented)

CI WXI H 4 a

KAC

3BC8 7 / . 11)- 1



Wet Density (pcf)	114.3
Dry Density (pcf)	94.9
% Moisture	20.5

Unconfined Compressive Strength (tsf)	2.55

CPFRPIKPHG 4 RO STHUURP BHUW
2 AB7 5 / . 00)- 0

Project Name: Mid Western Regional Sewer
Project Number: 2110.X
Sample: LWWTRF B7-3c   Depth: 16.0-16.5'
Sample Description: Sandy Lean CLAY, dark yellowish brown
Date: 10/22/2012
Tested By: B. Moore

blackburn consulting

Original Sample Length	6.00
Original Diameter (in)	2.40
Height-to-Diameter Ratio	2.5 : 1
Sample Area (in^2)	4.52

Test Results

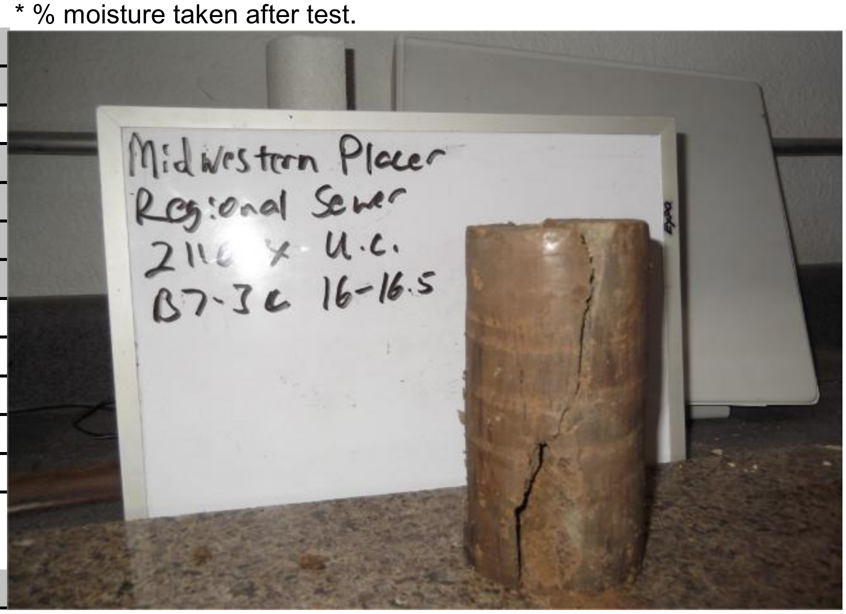
Axial Strain at Max. Load	7.8%
Average cross-sectional area (in^2)	4.91
Deflection at Max. Load (in)	0.470
Maximum Load (lbs)	177
Strain at Failure (%)	2.82
Compressive Strength (tsf)	/ +0-

Moisture Density

= HO DTMU1

* % moisture taken after test.

Tube and Sample (g)	921.70
Tube (g)	0.00
Sample Weight (g)	921.70
Tare Number	A1
Tare Weight (g)	154.80
Wet Weight (g)	473.60
Dry Weight (g)	419.70
Dry Weight (g)	264.90
Water Weight (g)	53.90
Percent Moisture (%)*	20.3
Wet Density (pcf)	129.4
Dry Density (pcf)	107.5



Compression Tests

Dial reading @ 0 lb	0.000

Rate of Strain=0.056in/min

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.003	2	0.328	150				
0.024	14	0.349	156				
0.044	29	0.369	161				
0.064	46	0.389	167				
0.084	60	0.409	171				
0.105	70	0.430	175				
0.125	80	0.450	176				
0.145	88	0.470	177				
0.166	95	0.491	175				
0.186	102	0.511	168				
0.207	109	0.531	151				
0.226	116	0.551	122				
0.247	122	0.571	98				
0.267	130	0.586	81				
0.287	137						
0.308	143						

: TRLHFW

Mid Western Regional Sewer

: TRLHFW8 XO EHT

2110.X

ADO SNH 8 XO EHT

LWWTRF B7-3c

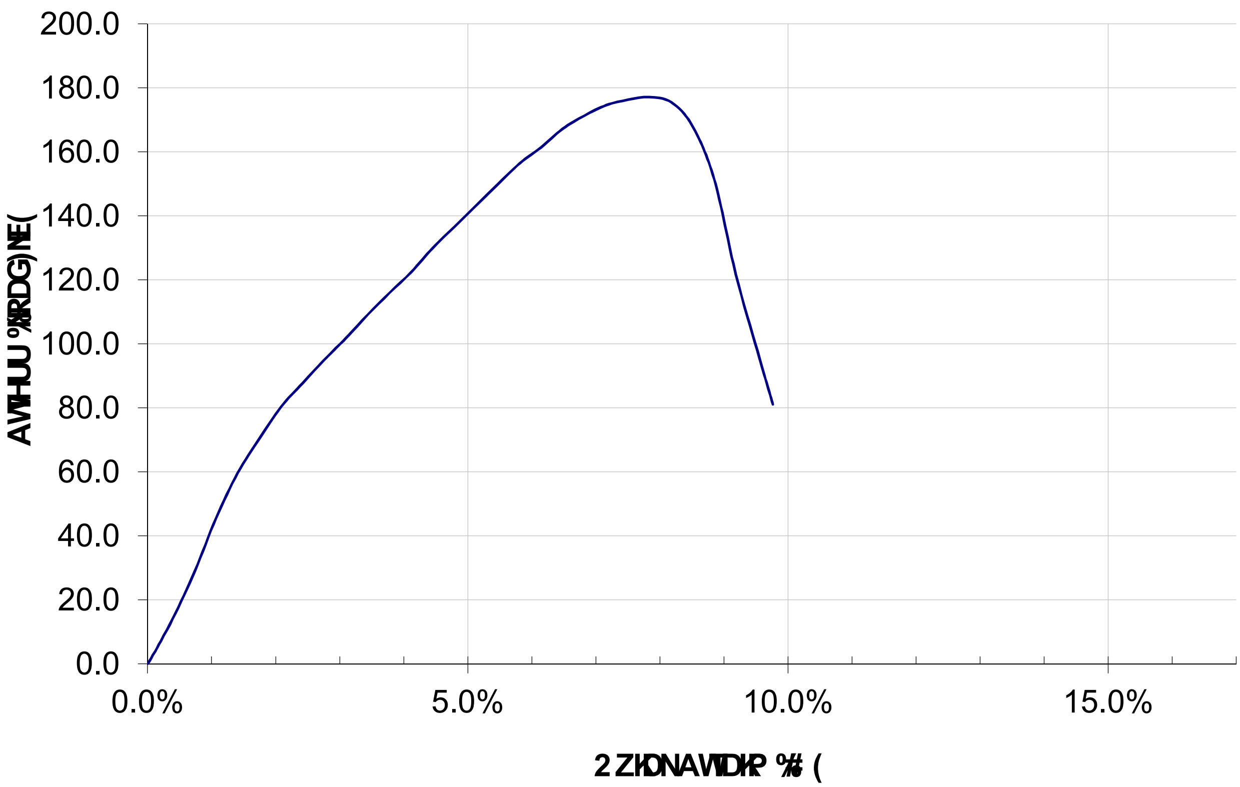
7 DWTHON5 HUFTHSWRP

Sandy Lean CLAY, dark yellowish brown

**BHUMG 3 **

B. Moore

2 AB7 5 / . 00)- 0



Wet Density (pcf)	129.4
Dry Density (pcf)	107.5
% Moisture	20.3

Unconfined Compressive Strength (tsf)	2.60

EXPANSION INDEX TEST

□r□□□□N□□ 2110.x	□□ □ Mid Wester Placer Regional Sewer	ASTM D4829-11
□□□ □□ LWWTRF B6-1B	D□TE 10/24/2012	□□ KLC

Initial Ht = 1 inches | G_s = 2.□ | *Factor* = $\frac{(4)(1728)(2.2046)}{(\pi)(4.01)^2(1000)}$ □ 0.3016

$$EI_{raw} = \frac{(1000)(\Delta H)}{H}$$

Dry Density (pcf) = $\gamma_d = \frac{\text{(Calc'd Dry Wt, gms) (Factor)}}{\text{(Sample ht. in inches)}}$

$$EI_{corrected} = EI_{raw} - \frac{(50-S)(65+EI_{raw})}{220-S}$$

$$\text{Saturation} = \frac{(100)(w)(Gs)(\gamma d)}{[(Gs)(62.4)]-\gamma d}$$

where: w □□ moisture in decimal
S □saturation in percent
H □initial height
ΔH = total change in height

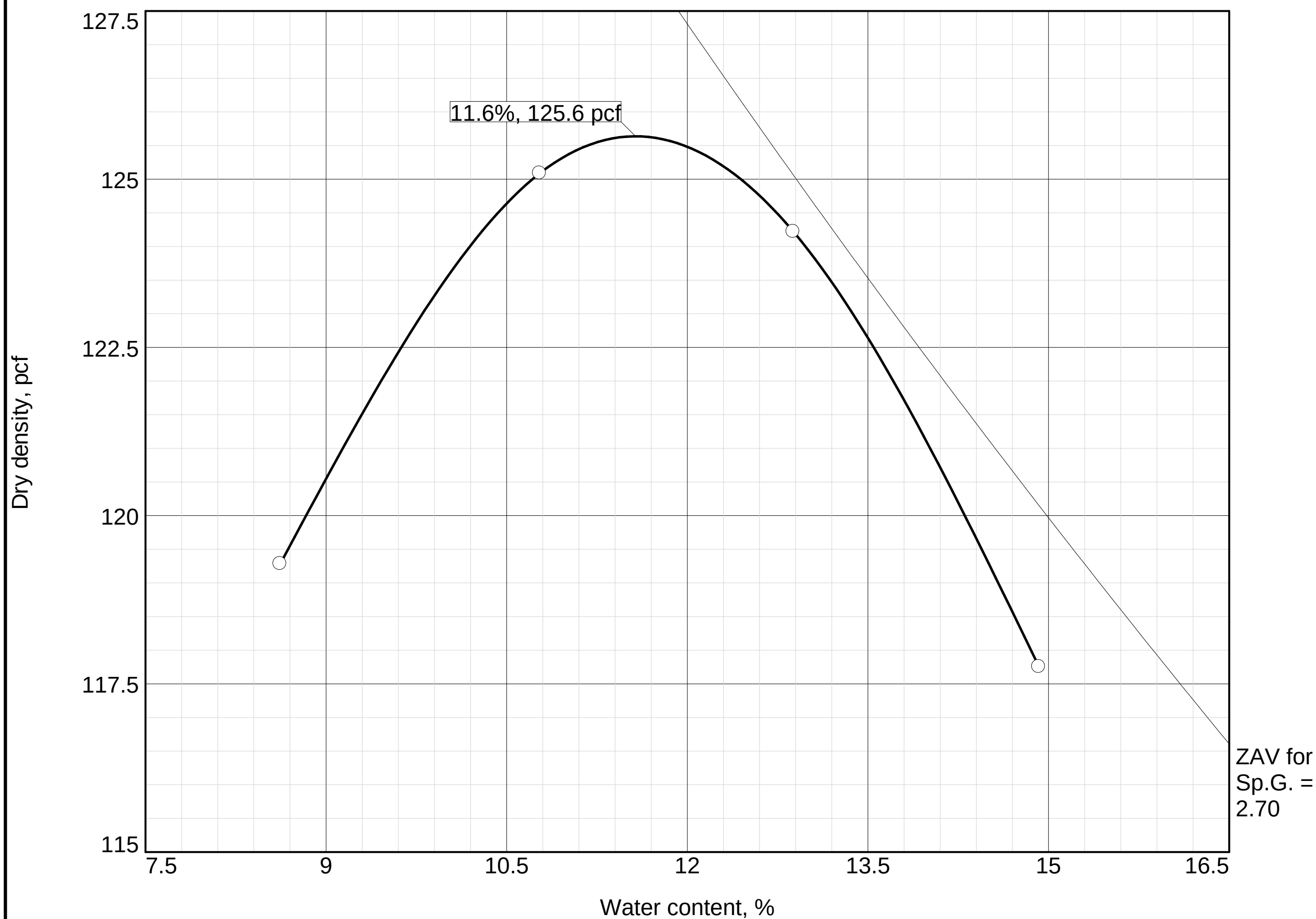
0 - 20	VERY LOW
21 - 50	LOW
51 - 90	MEDI□M
91 - 130	HIGH
□130	VERY HIGH

TRIAL 1						TRIAL 2					
DATE	TIME	LOAD	DIAL READ	REV CO□□T	TOTAL EXPA□	DATE	TIME	LOAD	DIAL READ	REV CO□□T	TOTAL EXPA□
DR□						**DR□**					
25-Oct	□:25	1 lb/in□2	0.1116	0	0.0000						
25-Oct	□:35	1 lb/in□2	0.1113	0	0.0000						
WET						**WET**					
25-Oct	□:3□	1 lb/in□2	0.0935	0	0.01□8						
25-Oct	□:55	1 lb/in□2	0.102	0	0.0093						
25-Oct	8:21	1 lb/in□2	0.102□	0	0.0086						
25-Oct	9:40	1 lb/in□2	0.1032	0	0.0081						
25-Oct	10:55	1 lb/in□2	0.1035	0	0.00□8						
25-Oct	14:16	1 lb/in□2	0.1036	0	0.00□□						
25-Oct	6:30	1 lb/in□2	0.1038	0	0.00□5						
25-Oct	□:30	1 lb/in□2	0.1038	0	0.00□5						

TRIAL 1						TRIAL 2					
Moisture Content			Density			Moisture Content			Density		
	Before	After		Before	After		Before	After		Before	After
Tare □o.	T11			R1		Tare □o.					
Gross Wet Wt (gm)	439.9	568.0	Wet□ ring (gms)	□63.8		Gross Wet Wt (gm)			Wet□ ring (gms)		
Gross Dry Wt (gm)	423.□	516.3	Ring (gms)	36□.1		Gross Dry Wt (gm)			Ring (gms)		
Water Loss (gm)	16.2	51.□	Wet Soil (gms)	396.□		Water Loss (gm)			Wet Soil (gms)		
Tare Wt. (gm)	258.1	306.6	Calc□d dry soil (gms)	361.4	361.4	Tare Wt. (gm)			Calc□d dry soil (gms)		
□et Dry Wt (gm)	165.6	209.□	Dry Dens (pcf)	109.0	10□.1	□et Dry Wt (gm)			Dry Dens (pcf)		
□ Moisture	9.8	24.□				□ Moisture					

Calculated Saturation (□)	48.4	116.1	Calculated Saturation (□)		
Total Swell (□)	1.8		Total Swell (□)		
Expansion Index (raw)	18		Expansion Index (raw)		
Expansion Index (corrected)	1□		Expansion Index (corrected)		

COMPACTION CURVE REPORT



Test specification: ASTM D 1557-07 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
0'-10.0'				2.70			2.0	

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 125.6 pcf Optimum moisture = 11.6 %	Yellowish Brown Sandy Lean CLAY

Project No. 2110.x **Client:** Stantec

Project: Mid Western Placer Regional Sewer

○ **Depth:** 0'-10.0' **Sample Number:** LWWTRF-5-Bag A

Remarks:
Sampled 9-25-2012
Specific Gravity estimated at 2.70

Blackburn Consulting

Auburn, CA

Figure

Tested By: KLC **Checked By:** RP

BSK Associates
Engineers & Laboratories

MINIMUM RESISTIVITY OF SOILS

Caltrans Test Method 643

1415 Tuolumne St.
Fresno, CA 93706
Ph: (559) 497-2868
Fax: (559) 485-6140

Project Name: Blackburn Consulting **Report Date:** 10/29/2012
Project Number: G10-085-10F **PO:** 10306 **Sample Date:** 9/25/2012
Lab Tracking ID: F12-544 **Test Date:** 10/22/2012
Sample Location: 2110.x Midwestern Placer Regional Sewer / LWWTRF B2-Bag B
Sample Description: Sandy Clay (CL) fine-medium grained, yellow-brown
Sampled By: T. McCrea (Blackburn) **Tested By:** S.M.

Soil temperature at minimum resistance = 23.8 °C

Total Moisture Added (ml)	Meter Dial Reading	Multiplier Setting	Resistance Measured (ohms)	Resistivity (ohm-cm)
10	4.9	1,000	4,900	5,917
20	1.7	1,000	1,700	2,053
30	1.6	1,000	1,600	1,932
40	1.8	1,000	1,800	2,174
Minimum Resistivity at 15.5°C, Ohm-cm				1,930

Remarks:

Reviewed By: [signature] 10/30/12

BSK Associates Engineers & Laboratories

Certificate of Analysis

Isaac Chavarria
BSK Associates - Fresno
567 W Shaw, Suite B
Fresno, CA 93704

Report Issue Date: 10/26/2012 15:16
Received Date: 10/22/2012
Received Time: 09:36

Lab Sample ID: A2J1821-01
Sample Date: 09/25/2012 09:30
Sample Type: Other

Client Project: G10-085-10F/F12-544
Sampled by: Blackburn Consulting
Matrix: Solid

Sample Description: LWWTRF B2-Bag B Brown Sandy Lean Clay

General Chemistry

Analyte	Method	Result	RL	Units	RL Mult	Batch	Prepared	Analyzed	Qual
*Chloride, Cal Trans Extract	California Test 422	18	3.0	mg/kg	1	A212057	10/24/12	10/24/12	
*pH, Cal Trans Extract	California Test 643	7.7		pH Units	1	A212198	10/26/12	10/26/12	
*pH Temperature in °C		20.6							
*Sulfate as SO4, Cal Trans Extract	California Test 417	20	6.0	mg/kg	1	A212057	10/24/12	10/24/12	

BSK Associates
Engineers & Laboratories

1415 Tuolumne St.
Fresno, CA 93706
Ph: (559) 497-2868
Fax: (559) 485-6140

MINIMUM RESISTIVITY OF SOILS

Caltrans Test Method 643

Project Name: Blackburn Consulting **Report Date:** 10/29/2012
Project Number: G10-085-10F **PO:** 10306 **Sample Date:** 9/25/2012
Lab Tracking ID: F12-544 **Test Date:** 10/22/2012
Sample Location: 2110.x Midwestern Placer Regional Sewer / LWWTRF B5-5B
Sample Description: Sandy Clay (CL) fine-medium grained, yellow-brown
Sampled By: T. McCrea (Blackburn) **Tested By:** S.M.

Soil temperature at minimum resistance = 23.8 ^{0}C

Total Moisture Added (ml)	Meter Dial Reading	Multiplier Setting	Resistance Measured (ohms)	Resistivity (ohm-cm)
0	1.8	1,000	1,800	2,174
10	8.6	100	860	1,038
20	9.4	100	940	1,135
Minimum Resistivity at 15.5°C, Ohm-cm				1,040

Remarks:

Reviewed By: [signature]

BSK Associates Engineers & Laboratories

Certificate of Analysis

Isaac Chavarria
BSK Associates - Fresno
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Fresno, CA 93704

Report Issue Date: 10/26/2012 15:16
Received Date: 10/22/2012
Received Time: 09:36

Lab Sample ID: A2J1821-02
Sample Date: 09/25/2012 11:00
Sample Type: Other

Client Project: G10-085-10F/F12-544
Sampled by: Blackburn Consulting
Matrix: Solid

Sample Description: LWWTRF B5-5B Yellowish Brown Sandy Lean Clay

General Chemistry

Analyte	Method	Result	RL	Units	RL Mult	Batch	Prepared	Analyzed	Qual
*Chloride, Cal Trans Extract	California Test 422	24	3.0	mg/kg	1	A212057	10/24/12	10/24/12	
*pH, Cal Trans Extract	California Test 643	7.5		pH Units	1	A212198	10/26/12	10/26/12	
*pH Temperature in °C		21.4							
*Sulfate as SO4, Cal Trans Extract	California Test 417	8.3	6.0	mg/kg	1	A212057	10/24/12	10/24/12	

BSK Associates Engineers & Laboratories

1415 Tuolumne St.
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Ph: (559) 497-2868
Fax: (559) 485-6140

MINIMUM RESISTIVITY OF SOILS

Caltrans Test Method 643

Project Name: Blackburn Consulting

Project Number: G10-085-10F **PO:** 10306 **Report Date:** 10/29/2012

Lab Tracking ID: F12-544 **Sample Date:** 9/25/2012

Sample Location: 2110.x Midwestern Placer Regional Sewer / LWWTRF B7-3B **Test Date:** 10/22/2012

Sample Description: Sandy Clay (CL) fine-medium grained, brown

Sampled By: T. McCrea (Blackburn) **Tested By:** S.M.

Soil temperature at minimum resistance = 24.1 °C

Total Moisture Added (ml)	Meter Dial Reading	Multiplier Setting	Resistance Measured (ohms)	Resistivity (ohm-cm)
0	6.1	1,000	6,100	7,412
10	1.2	1,000	1,200	1,458
20	1.0	1,000	1,000	1,215
30	1.3	1,000	1,300	1,580
Minimum Resistivity at 15.5°C, Ohm-cm				1,220

Remarks:

Reviewed By: [signature] 10/30/12

BSK Associates Engineers & Laboratories

Certificate of Analysis

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BSK Associates - Fresno
567 W Shaw, Suite B
Fresno, CA 93704

Report Issue Date: 10/26/2012 15:16
Received Date: 10/22/2012
Received Time: 09:36

Lab Sample ID: A2J1821-03
Sample Date: 09/25/2012 14:00
Sample Type: Other

Client Project: G10-085-10F/F12-544
Sampled by: Blackburn Consulting
Matrix: Solid

Sample Description: LWWTRF B7-3B Brown Sandy Lean Clay

General Chemistry

Analyte	Method	Result	RL	Units	RL Mult	Batch	Prepared	Analyzed	Qual
*Chloride, Cal Trans Extract	California Test 422	28	3.0	mg/kg	1	A212057	10/24/12	10/24/12	
*pH, Cal Trans Extract	California Test 643	7.7		pH Units	1	A212198	10/26/12	10/26/12	
*pH Temperature in °C		20.6							
*Sulfate as SO4, Cal Trans Extract	California Test 417	10	6.0	mg/kg	1	A212057	10/24/12	10/24/12	

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
WWTP Improvements
Placer County, CA

APPENDIX C

Important Information About
This Geotechnical Engineering Report,
Geoprofessional Business Association

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, CA

Prepared by:

BLACKBURN CONSULTING
11521 Blocker Drive, Suite 110
Auburn, CA 95603
(530) 887-1494

April 2018

Prepared for:

Stantec
3875 Atherton Road
Rocklin, CA 95765

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blackburn consulting

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Geotechnical ▪ Geo-Environmental ▪ Construction Services ▪ Forensics

File No. 3228.X
April 10, 2018

Mr. Gabe Aronow, P.E.
Stantec
3875 Atherton Road
Rocklin CA 95765

Subject: **GEOTECHNICAL DESIGN REPORT**
Lincoln Wastewater Treatment and Reclamation Facility Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, California

Dear Mr. Aronow:

Blackburn Consulting (BCI) is pleased to submit this Geotechnical Design Report for the Lincoln Wastewater Treatment and Reclamation Facility Phase 1 Expansion Project, Tertiary Storage Basin No. 3, located in Placer County, California. BCI prepared this report in accordance with our June 6, 2017 agreement.

This report presents geotechnical and geologic data and provides recommendations to design and construct the new basin.

Please call us if you have questions or require additional information.

Sincerely,

BLACKBURN CONSULTING

PROFESSIONAL GEOLOGIST ROBERT C. PICKARD No. 2508 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER THOMAS W. BLACKBURN No. 2311 Exp. GEOTECHNICAL STATE OF CALIFORNIA

Rob Pickard, P.G., C.E.G
Project Engineering Geologist

Thomas W. Blackburn, G.E., P.E.
Senior Principal

GEOTECHNICAL DESIGN REPORT
Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, CA

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GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, CA

FIGURES

APPENDIX A

APPENDIX B

APPENDIX C

1 INTRODUCTION

1.1 Purpose

Blackburn Consulting (BCI) prepared this Geotechnical Memorandum for the planned third Tertiary Storage Basin included in the Phase 1 Expansion Project at the City of Lincoln Wastewater Treatment and Reclamation Facility located in Placer County, California.

BCI prepared this report for design and construction of the proposed embankments for the new tertiary storage basin. Do not rely upon this report for different locations or improvements without the written consent of BCI.

1.2 Scope of Services

To prepare this report, BCI:

- Discussed the proposed Tertiary Storage Basin No. 3 (TSB No. 3) with Stantec,
- Reviewed published geologic mapping,
- Reviewed available geotechnical reports for existing facilities, including:
    - Carlton Engineering, August 1999, Remote Storage Basins, East of Fiddyment Road, Placer County, California.
    - Kleinfelder, March 2001, Geotechnical Investigation Report.
    - Kleinfelder, January 2002, Updated Geotechnical Investigation Report.
    - BCI, April 2013, Geotechnical Design Report, Mid-Western Placer Regional Sewer Project.
    - BCI, November 2017, Geotechnical Design Report, Lincoln Wastewater Treatment and Reclamation Facility Phase 1 and 2 Expansion Project.
- Reviewed plans for the existing tertiary storage basins, dated 1999 and 2006,
- Reviewed plans for the existing emergency storage basin, dated 2001 and 2003,
- Performed field investigation and laboratory analyses,
- Performed engineering analysis and calculations.

1.3 Site Location and Description

The expansion project is located in an unincorporated area of Placer County. Figure 1 shows the project location.

The project adds a third tertiary storage basin at the City of Lincoln Wastewater Treatment and Reclamation Facility (WWTRF). Figure 2 shows the approximate location of the third basin.

1.4 Project Description

Stantec's proposed design indicates that TSB No. 3 will:

- Hold approximately 80 million gallons,
- Have 24-foot high, homogeneous, blended soil embankments (no zones or cores) built to 3h:1v slopes on both water (inner) and land (outer) sides,
- Have a "berm" on the outer side of the south and west embankments built up to an elevation of about 110 feet,
- Have embankment crest elevations around 125 feet and bottom of basin elevations around 101 feet,
- Have piping and associated vaults installed in the northeast corner of the existing embankment.
- Be fully lined with an HDPE liner, to mitigate through seepage and underseepage.

Stantec has designated borrow sites on the north and east sides of the proposed TSB No. 3 to construct the south and west embankments of the new basin (see Figure 2). The existing south embankment of the Emergency Storage Basin (ESB) will form the north embankment of the new basin. The existing west embankment of Tertiary Storage Basin No. 2 (TSB No. 2) will form the east embankment of the new basin. The borrow excavation will increase the height of these two existing embankments from about 10 to 15 feet to about 21 to 24 feet, measured from the crest to the toe of the inner slope. Additionally, approximately 14- inches of fill will be added to the top of the existing northern embankment.

Figure 2 shows the approximate embankment location and borrow areas.

2 GEOLOGIC CONDITIONS

2.1 General Geology

Our site work and published geologic mapping[1] show the site is underlain by Quaternary deposits of the Riverbank Formation.

The Riverbank Formation is an alluvial deposit typically composed of interbedded medium dense to dense sands, often cemented, and stiff to hard silts and clays. Bedding is typically horizontal, lenticular, and discontinuous. These sediments were deposited in the Late Pleistocene age (deposited over 150,000 years ago). This unit is shown as "Qrl" and "Qru" (Lower and Upper Riverbank) on Figure 3. Our exploratory borings and test pits confirm that the site is underlain by interbedded clays, silts, and sands, which is consistent with the Riverbank Formation.

[1] Helley, E.J. and Harwood, D.S., 1985, Geologic Map of the Late Cenozoic Deposits of the Sacramento Valley and Northern Sierra Foothills: U.S. Geological Survey, Map MF-1790.

2.2 Faulting

The Fault Activity Map of California[2] does not identify Historic or Holocene age faults (displacement within the last 11,700 years) within or adjacent to the project site. The nearest mapped fault is the Cleveland Hill Fault located approximately 40 miles north of the site. Figure 4 shows the approximate location of faulting in the region.

3 FIELDWORK AND LABORATORY TESTS

3.1 Exploratory Borings and Test Pits

To characterize the subsurface conditions, BCI drilled, logged, and sampled six borings (B1 through B6) on October 6, 2017, and eight test pits (TP1 through TP8) on October 31, 2017. Borings extended to 26.5 feet below existing ground surface, and test pits extended 6.5 to 9.0 feet below existing ground surface. Figure 2 shows the approximate boring and test pit locations. We include logs of the explorations in Appendix A.

We located exploration points using a handheld GPS and geographic features shown on the project topographic mapping.

Our subcontractor, Taber Drilling, drilled the borings using 4-inch solid-stem auger. We obtained soil samples at various intervals using a 3.0-inch O.D. Modified California (MC) sampler (equipped with 2.4-inch diameter brass liners), driven with an automatic hammer, weighing 140-pounds and falling approximately 30 inches.

Our subcontractor, Rob Rasch, excavated test pits using a Bobcat E32.

A BCI engineer logged the borings and test pits and retrieved samples for laboratory testing. We used plastic caps to seal and label the 2.4-inch diameter, 6-inch long brass tubes retrieved from MC sampling. We also retrieved bulk soil samples from auger cuttings at varied depths, placed this material in large plastic bags, and labeled them for laboratory identification. Similarly, we took bulk samples from each soil type identified in the test pits and placed the samples in large plastic bags to be used for laboratory analysis.

During our field exploration, we performed field strength testing with a pocket penetrometer on select cohesive and/or cemented soil samples. We note the results of field tests on the boring logs.

[2] Jennings, Charles W., and Bryant, William A., 2010 Fault Activity Map of California: California Geological Survey, Geologic Data Map No. 6.

3.2 Laboratory Testing

We completed the following laboratory tests on representative soil samples from our exploratory borings:

- Moisture content and unit weight to classify and characterize the in-place soil characteristics
- Plasticity index to classify the soil
- Sieve analysis to classify the soil
- Triaxial undrained, unconfined compression to estimate strength
- Direct shear to estimate strength
- Maximum dry density to estimate compaction characteristics

See Appendix B for a laboratory summary sheet and laboratory test results. We also include these results in our the boring and test pit logs in Appendix A.

4 SUBSURFACE FINDINGS

4.1 Soil Conditions

We encountered the following soil profile in our test pits and borings:

- Proposed borrow areas:
  - Approximate north side of TSB No. 3 above about elevation 100 feet: (B-1, B-2, TP-1, TP-2, TP-3): Mostly stiff to hard lean clays and medium dense clayey sands.
  - Approximate east side of TSB No. 3, above about elevation 100 feet (B-3, B-5, B-6, TP-4, TP-7, TP-8): Mostly medium dense clayey sands and very stiff to hard lean clays.
- Proposed foundation soils for embankments from about elevation 100 feet to 90 feet (TP-1, B-1, TP-3, B-3, B-6, B-5, TP-6, B-4, TP-5, B-2): Mostly stiff to hard lean clays and medium dense sands. We recorded pocket penetrometer tests on fine-grained (clay) soil samples mostly above 4.0 tons per square foot (tsf), with some zones ranging from 1.3 to 3.8 tsf (see logs) and triaxial undrained, unconfined (UU) compression test strengths from 1.15 to 3.01 tsf.

  The clayey sands are weakly to moderately cemented with pocket penetrometer tests at or above 4.5 tsf and direct shear strength tests with cohesion values ranging from 0 to 0.6 tsf and φ values of 33° to 39°.
- Underlying soils below approximate elevation 90 feet (B-1, B-2, B-3, B-4, B-6): Stiff to hard lean clays. We recorded pocket penetrometer tests on fine-grained (clay) soil samples mostly above 3.5 tsf.

Refer to the logs in Appendix A and laboratory tests in Appendix B for more specific subsurface conditions.

4.2 Groundwater

We encountered groundwater in the borings listed in Table 1. We did not encounter groundwater in any of our test pits, which were explored to depths of 6.5 to 9.0 feet bgs.

TABLE 1

Groundwater Summary	
Boring/Approximate Elevation (ft)	**Depth to Water/Approximate Elevation (ft)**
B2/107.5	15/92.5
B4/109	18/91

Groundwater at the facility has previously been recorded at shallower depths than what is shown above. Kleinfelder[3] recorded groundwater in their borings at depths ranging from 11.5 to 28.5 feet bgs (about elevation 99 to 82 ft) in March-April 2000. A monitoring well placed by Kleinfelder showed groundwater depths ranging from 13.0 ft in March 2000 to 16.9 feet in January 2001 (approximate elevations of 97.5 feet and 93.6 feet).

We recorded groundwater at depths ranging from 22.3 to 28.0 feet bgs (about elevation 88.2 to 82.5 feet) in our September 2012 borings[4]. It is not unusual to encounter sand lenses which can contain perched groundwater at varied depths within the Riverbank Formation.

For project design, assume a groundwater elevation of 99 feet. Groundwater may, on occasion, reach as high as the base of the new basin (elevation 101 feet). This level does not account for seepage from the adjacent basins. HDPE liners may be damaged when groundwater is close to, or above the bottom of the liner. For operation and maintenance, we recommend careful groundwater monitoring in the area TSB No. 3 (and the surrounding basins) to mitigate liner damage.

[3] Kleinfelder, 2002, Updated Geotechnical Investigation Report, Proposed Lincoln Wastewater Treatment Plant, Fiddyment Road, Placer County, California; consultant's report to Del Webb California Corporation
[4] BCI, 2013, Geotechnical Design Report, Midwestern Placer Sewer Project, Placer County, California.

5 EMBANKMENT STABILITY AND SEEPAGE ANALYSIS

We address possible embankment failure modes below:

- End of construction. This occurs on medium to tall earth embankments, when pore pressures build during construction and lower strengths. Given the low height of these embankments, proposed 3h:1v inner and outer slope gradients, and very stiff to hard clay foundation materials we do not expect failure from this condition.
- Rapid draw down of the basin. This occurs after an embankment becomes saturated, and the basin water level lowers so quickly that pore pressures in the embankment soils do not have time to dissipate. Since TSB No. 3 will be lined (assuming the HDPE liner is installed correctly and does not leak), the embankment soils should never become saturated from steady state seepage, and so rapid drawdown is not a consideration for TSB No. 3.
- Steady State Condition. We modeled the embankments using the for both static and pseudostatic conditions using Stantec's design slopes with an HDPE liner.

5.1 Cross-section Development for Analysis

To analyze embankment stability, we selected two embankment cross-sections based on our review of the existing topography, subsurface conditions, and preliminary drawings for the new basin provided by Stantec.

Our first cross-section represents the north and east embankments of the new basin (embankments shared with the ESB and TSB No. 2). Table 2 shows the soil properties used for our analysis of this cross-section.

TABLE 2

North and East Embankment Cross-section			
Soil Description	**ϕ'**	**c', psf**	**Unit weight, γ, pcf**
Existing embankment fill, sandy lean clay, and clayey sands to elev. 109 ft	32°	50	129
Native clayey sands, elev. 91 to 109 ft or	35°	110	126
Native sandy clays and lean clays, elev. 91 to 109 ft	0°	2000	126
Native sandy clays and lean clays below elev. 91 ft	0°	2000	122

Our second cross-section represents the south and west embankments. Table 3 shows the soil properties we used for this case.

TABLE 3

South and West Embankment Cross-section			
Soil Description	**ϕ'**	**c', psf**	**Unit weight, γ, pcf**
Embankment fill: sandy lean clay and clayey sands to elev. 98 to 101.5 ft	32°	50	129
Engineered fill: sandy lean clay and clayey sands, elev. 98 to 110 ft	31°	25	129
Native clayey sands, elev. 90 to 101.5 ft or	35°	110	126
Native sandy clays and lean clays, elev. 90 to 101.5 ft	0°	2000	126
Native sandy clays and lean clays below elev. 90 ft	0°	2000	122

As indicated in Tables 3 and 4, we evaluated both cross-sections for either a sandy clay upper foundation layer, or for a clayey sand upper foundation layer, based on variations observed in our exploratory borings and test pits. We discuss our analysis results in section 5.2, below.

We made the following assumptions in our analysis:

- Where borrow material is excavated along an existing embankment, slopes will continue at their existing angle (3h:1v and 2.5h:1v)
- New embankments will have slope gradients of 3h:1v on both sides, with a crest width of 12 feet.
- The pore pressures in the embankment will not be affected by the water held in the basin because the basin will be fully lined.
- For the north and east embankments, we conservatively assume that the ESB and TSB No. 2 have a water surface elevation at the top of the embankment when evaluating the inner slope of TSB No. 3, and that they are empty when evaluating the outer slope of TSB No. 3,
- For all embankments, we conservatively assume that TSB No. 3 has a water surface elevation at the top of the embankment when evaluating its outer slopes,
- We modeled groundwater at elevation 101 feet, based on a conservative evaluation of measured groundwater in the region.

Figures 4 through 8 show our model cross sections as described above.

5.2 Analysis Methodology and Results

BCI used the program SLOPE/W, version 7.23, to perform slope stability analysis.

For long term slope stability analysis, we used:

- The Spencer limit-equilibrium method of analysis,
- Profile representing the maximum crest height and lowest toe elevation for each embankment analyzed,
- Soil profile and strength characteristics as discussed in section 5.1, using a clayey sand foundation layer (most conservative),
- Pore pressures based on an assumed groundwater surface elevation of 101 feet,
- A tension crack search, which prevents the application of unrealistic tensile strengths in the clay embankment.

Table 4 summarizes our slope stability analysis results.

TABLE 4

Slope Stability Results			
Location	**Water Surface Condition**	**Steady-State Slope Stability Factor of Safety**	**Shown on Figure**
North or East Embankment, Inner Slope	Emergency Storage Basin/Tertiary Storage Basin No. 2 full, Tertiary Storage Basin No. 3 empty	2.05	5
North or East Embankment, Outer Slope	Emergency Storage Basin/Tertiary Storage Basin No. 2 empty, Tertiary Storage Basin No. 3 full	2.39	6
South and West Embankment, Inner Slope	Tertiary Storage Basin No. 3 empty	2.25	7
South or West Embankment, Outer Slope	Tertiary Storage Basin No. 3 full	2.38	8

In general, higher risk structures such as earthen dams and levees are required to show minimum factors of safety against static slope failure of 1.4[5,6,7] to 1.5[8].

We evaluated seismic vulnerability using a pseudostatic analysis with:

- The Bray & Travasarou method[9] to calculate the seismic coefficient,
- A moment magnitude of 6.5,
- The same critical water surfaces shown in Table 5, above,
- The Spencer limit-equilibrium method of analysis,
- Soil profile and strength characteristics as discussed in section 5.1,
- Pore pressures based on an assumed groundwater surface elevation of 101 feet,
- A tension crack search, which prevents the application of unrealistic tensile strengths in the clay embankment.

We calculated seismic coefficients that range from 0.123 to 0.176. The coefficient calculation is based on site specific parameters and a 16% probability of a seismic displacement greater than 4 inches (vertical). For slope stability analysis using Slope/W we used a conservatively applied a seismic coefficient of 0.2. We calculated factors of safety over 1.2 for each section analyzed. Since the calculated factors of safety are greater than 1.0, we conclude there is less than a 16% probability that a seismic displacement of the embankments would exceed 4 inches.

5.3 Steady State Seepage Analysis

Steady State Seepage occurs when a basin fills and partially saturates an embankment. Since TSB No. 3 will be fully lined, we don't expect seepage through the embankments and therefore did not analyze this condition.

[5] CA Department of Water Resources, Urban Levee Design Criteria, May 2012
[6] URS for CA Department of Water Resources, Guidance Document for Geotechnical Analyses, Urban Levee Evaluations Project, April 2015
[7] USACE, Engineering Manual 1110-2-1913: Design and Construction of Levees, April 2000.
[8] USACE, Engineering Manual 1110-2-1902: Slope Stability, October 2003
[9] Bray, Jonathan, and Travasrou, Thaleia, September 2009, Pseudostatic Coefficient for Use in Simplified Seismic Slope Stability Evaluation, ASCE Journal of Geotechnical and Geoenvironmental Engineering.

6 CONCLUSIONS AND RECOMMENDATIONS

We base our recommendations on an impermeable (HDPE lined) basin.

6.1 Embankment Design

Based on the results of our analysis we recommend the following embankment geometry:

- Minimum crown width of 10 feet,
- New interior and exterior fill slope gradients of 3h:1v,
- Extensions of existing slopes (cut) can be cut to match existing gradients (2.5 to 3h:1v).

6.2 Ground preparation and Keyway

Clear all debris and/or obstructions above the ground surface. This includes all brush and vegetation, as well as any structures to be abandoned.

Widen and remove all loose soil from all depressions/trenches made by vegetation and/or structure removal to allow for subsequent backfilling and compaction equipment.

Flatten the sides of all holes and depressions caused by the clearing and grubbing operations before backfilling. Backfill with materials similar to adjacent soils and place in compacted layers to grade.

Where borrow material has already been recently removed (anywhere below an elevation of approximately 105 feet), no keyway is required (organic material at the surface will still need to be removed).

Where the existing ground elevation is above 105 feet, over-excavate a 2-foot deep, minimum 10-foot-wide key centered under the embankment for foundation soil inspection and improved shear resistance. Retain BCI to observe the key for loose/soft soil or unsuitable materials.

Prior to placement of fill, scarify the ground surface to a minimum depth of 6 inches. Moisture condition the scarified soil to within 2% of optimum and compact to minimum of 90% of ASTM D 1557 test procedure.

6.3 Embankment Fill Requirements

The borrow site material should be suitable for embankment fill, provided that the contractor removes organics and any material greater than 3 inches in diameter.

Import fill should meet the following criteria:

- 100% passing the 3-inch sieve
- 90% to 100% passing the 2-inch sieve
- 75% to 100% passing the No. 4 sieve
- 20-60% passing the No. 200 sieve
- Liquid Limit ≤ 45
- Plasticity Index ≥ 8 and ≤ 30
- Shall not contain organics, debris or other deleterious material
- Approval from BCI prior to placement

Place fill in maximum 8-inch thick loose lifts, moisture condition to 1% to 2% above optimum, and compact to a minimum of 90% relative compaction based on ASTM D 1557 test procedure. Compact fill using a sheepsfoot or padded drum type roller.

Where fill is placed on sloping ground, blade back slopes horizontally during placement of embankment fill to create a stepped (or benched) fill surface (such that a uniform, sloping fill surface is avoided). Benching must remove loose surficial soils and result in stepped benches, generally one to two feet in height and depth into the existing slope. The lower bench should be sloped a minimum of 2% into the slope. Where benching will interfere with existing structures, utilities, or vegetation, BCI can review modifications on a case-by-case basis.

Construct fill slopes no steeper than 2(H):1(V). To achieve adequate compaction on the face of fill slopes, over-build the slopes and then cut back to the design grade. Track-walking is not an adequate method to compact the face of slopes.

Use the above embankment fill requirement for construction of the “berm”.

6.4 Inlet/Outlet Pipe Installation

We anticipate that inlet and outlet pipes will be included in the final design of the new basin. We expect the pipes and outlet structure will be constructed within native, very stiff to hard or medium dense to dense clays and sands.

We expect adequate foundation support for pipes placed in native soil and that settlement will be negligible following proper placement and backfill. We expect trench excavations to be relatively stable. For preliminary consideration, use a Type A soil classification (Federal Register, OSHA, 29 CFR Part 1926) for trench sloping and/or shoring design. Excavations may encounter clayey or clean sands, or groundwater, in which case sloping/shoring will need to be modified for a Type C soil classification. Final sloping/shoring based on actual conditions is the responsibility of the contractor.

For pipe beneath the basin embankment, construct in accordance with the following:

- Best option: Use controlled, low strength material (CLSM) to backfill and encapsulate the pipe (which also allows a narrower trench).

Or:

- Bring embankment fill up to a grade of approximately 2 feet above the crown of the pipe prior to excavation for the pipe. Excavate the trench to a depth of approximately 2 feet below the bottom of the pipe and at least 4 feet wider than the pipe.
- Selectively stockpile material so the contractor can be reuse it as backfill.
- After the contractor excavates the trench, backfill it to the pipe invert elevation. Compact the backfill with mechanical compactors to a minimum of 90% percent relative compaction near optimum moisture content.
- Bring backfill up evenly on both sides of the pipe to avoid unequal side loads that could fail or move the pipe. Take special care in the vicinity of any protrusions such as joint collars to achieve proper compaction.

6.5 Structures in Embankments

Stantec plans (dated 3/7/2018) show two approximately 10 foot diameter vaults in the northeast corner TSB No. 3 in the existing embankment These are below-grade structures and the net pressure exerted upon the subsurface will be similar to or less than the current load. Excavation for below-grade structures reduces the net pressure by removing soil that acts as a "preload" to the underlying soils, thus "unloading" the bearing materials before "loading" by placement of the structure.

We anticipate the vaults will be founded on native soils and will use a mat type foundation for support. Based on these assumptions:

- Use a maximum net contact pressure for vault mat foundation of 1,500 psf.
- Expect settlement of mat foundations less than 1 inch with differential settlement less than ½-inch across the pump station structure.
- Clean footing excavations of debris and loose soil prior to placing concrete.
- BCI must observe all footing excavations prior to reinforcement placement to verify competent bearing materials.

- For subgrade uniformity, use Caltrans Class 2 aggregate baserock as underlayment (this is not geotechnically necessary provided a firm uniform subgrade is obtained). If an aggregate underlayment is used, place a minimum thickness of 6-inches and compact to a minimum of 95% relative compaction (per ASTM D 1557 test method).
- Crushed rock underlayment may also be used (and can benefit excavation dewatering). Underlay the crushed rock with a geotextile filter fabric (ie. Mirafi 140N) and compact the rock with at least 6 passes of a static roller.

Since TSB No. 1, which is not lined, is adjacent to the NE corner of TSB No. 3, we recommend using undrained shear strengths. For evaluation of lateral earth pressures, use the undrained backfill with level ground conditions equivalent fluid weights (EFW) shown in the Table 5 below.

TABLE 5

LATERAL EARTH PRESSURES	
Condition	**Undrained Equivalent Fluid Weight (pcf)**
At-Rest	100
Active	86
Passive	270 (F.S. = 1)
Seismic (Active and At-Rest)	6

The above pressures assume structure backfill placed against the structure wall in accordance with our recommendations, and a saturated unit weight of approximately 133 pounds per cubic foot (pcf). Notify BCI if these assumptions are not valid so that we may assess the situation and provide additional recommendations, if necessary. Backfill with CLSM is an acceptable alternative.

For seismic loading, add the Seismic EFW to the at-rest or active EFW and apply the total force as a uniform load on the wall with a resultant located at 0.4H where H is the backfill height. We estimated the EFWs for seismic loading using the Mononobe-Okabe equation and a horizontal seismic acceleration coefficient, k_h, of approximately ½ the expected PGA. This k_h value assumes that the walls displace at least 1-inch during the design seismic event.

Surface loads (footings, storage, vehicle traffic) applied near the wall will increase the lateral pressure on the wall. A uniform surface load of 200 psf to 300 psf is often used to approximate construction traffic loading on walls. In general, if surface loads are closer to the edge of the retaining wall than three-fourths of the retained height, increase the design wall pressure by 0.5q over the area of the retaining wall. In this expression, q is the surface surcharge load in psf. This is a conservative procedure and lower design pressures may be applicable upon evaluation of individual surface loads and setback distances.

6.6 Embankment Liner

Stantec has not yet selected the final liner but we expect design and placement (subgrade preparation, bedding, drainage, etc.) to be in accordance with the manufacturer's recommendations. BCI can provide supporting information if necessary as addendum information to this report.

Groundwater can perch above the underlying soil in the area of the basin. Liner design should include considerations for groundwater buildup and drainage.

6.7 Erosion Control

The outer exposed slopes are subject to erosion. To reduce erosion rills and gulleys, use hydroseeding and/or erosion control surfacing to protect exterior slopes. If there is not adequate time for standard hydroseeding to become established before heavy rains are likely, use an erosion control blanket (such as Landlok® S2 or an equivalent) or a bonded, hydraulically applied blanket (such as Flexterra® FGM or an equivalent). Expect future maintenance, such as periodic addition of slope protection, slope re-grading, or occasional reworking and/or re-compaction of the exposed surfaces

6.8 Dewatering

If construction proceeds in the late summer and fall months we do not anticipate groundwater will affect construction. If localized perched groundwater is encountered, well points will likely work best to cut off flow into the excavation by drawing down the water level over a large area. We recommend that if required, groundwater should be drawn down at least 5 feet below the planned bottom of excavation.

7 RISK MANAGEMENT

Our experience and that of our profession clearly indicates that the risks of costly design, construction, and maintenance problems can be significantly lowered by retaining the geotechnical engineer of record to provide additional services during design and construction.

For this project, we recommend that the project owner retain us to:

- Review and provide comments on the civil plans and specifications prior to construction.
- Monitor construction to check and document our report assumptions. At a minimum, BCI should observe excavations, approve backfill, observe and test placement and compaction of fill for embankments and structures.
- Update this report if design changes occur, 2 years or more lapses between this report and construction, and/or site conditions have changed.

If we are not retained to perform the above applicable services, we are not responsible for any other party's interpretation of our report, and subsequent addendums, letters, and discussions.

8 LIMITATIONS

BCI performed services in accordance with generally accepted geotechnical engineering principles and practices currently used in this area. Where referenced, we used ASTM and California Test Method standards as a general (not strict) guideline only. Do not use or rely upon this report for different locations or improvements without the written consent of BCI.

We do not warranty our services.

BCI based this report on the current site conditions. We assume our boring and test pit soil and groundwater conditions are representative of the subsurface conditions throughout the site. Conditions at locations other than our explorations could be different.

Appendix A shows logs of our explorations. The lines designating the interface between soil types are approximate. The transition between material types may be abrupt or gradual. We based our recommendations on the final logs, which represents our interpretation of the field log and general knowledge of the site and geological conditions. We based our boring and test pit log descriptions on our field logging, geologic mapping, and laboratory testing.

The groundwater elevations discussed in this report represent the groundwater elevation during the time of our subsurface exploration, at the specific exploration locations, and groundwater observed by others. The groundwater table may be lower or higher in the future – which may damage the TSB No. 3 liner. Consider potential groundwater levels in planning operation and maintenance of the basins.

Modern design and construction are complex, with many regulatory sources/restrictions, involved parties, construction alternatives, etc. It is common to experience changes and delays. The owner should set aside a reasonable contingency fund based on complexities and cost estimates to cover changes and delays.

Appendix C shows GBA guidelines for how to use this report.

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility

Phase 1 Expansion

Tertiary Storage Basin No. 3

Placer County, CA

FIGURES

Vicinity Map

Site Map

Regional Geologic Map

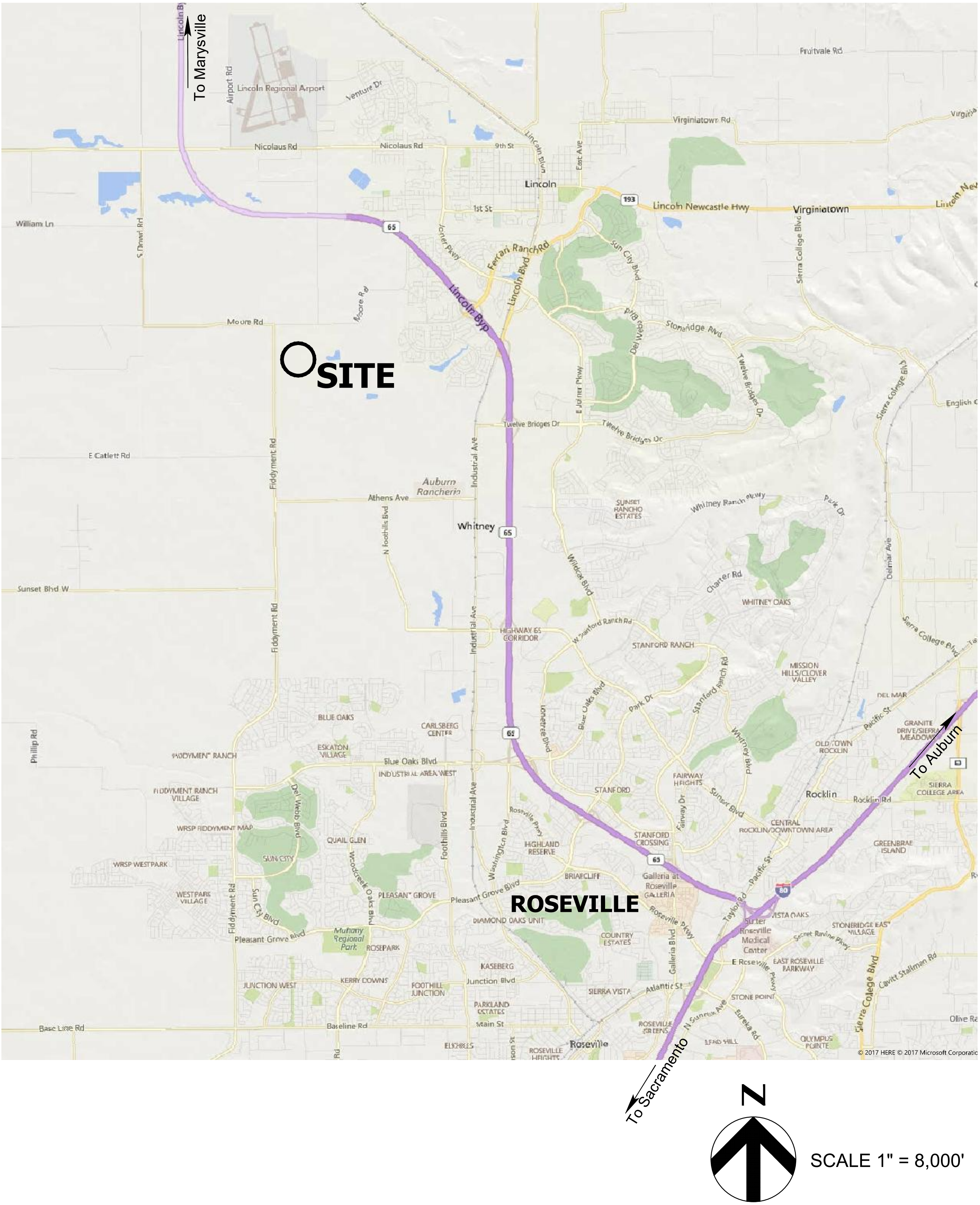
Regional Fault Map

North and East Embankment Cross-Section, Inner Slope

North and East Embankment Cross-Section, Outer Slope

South and West Embankment Cross-Section, Inner Slope

South and West Embankment Cross-Section, Outer Slope



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VICINITY MAP

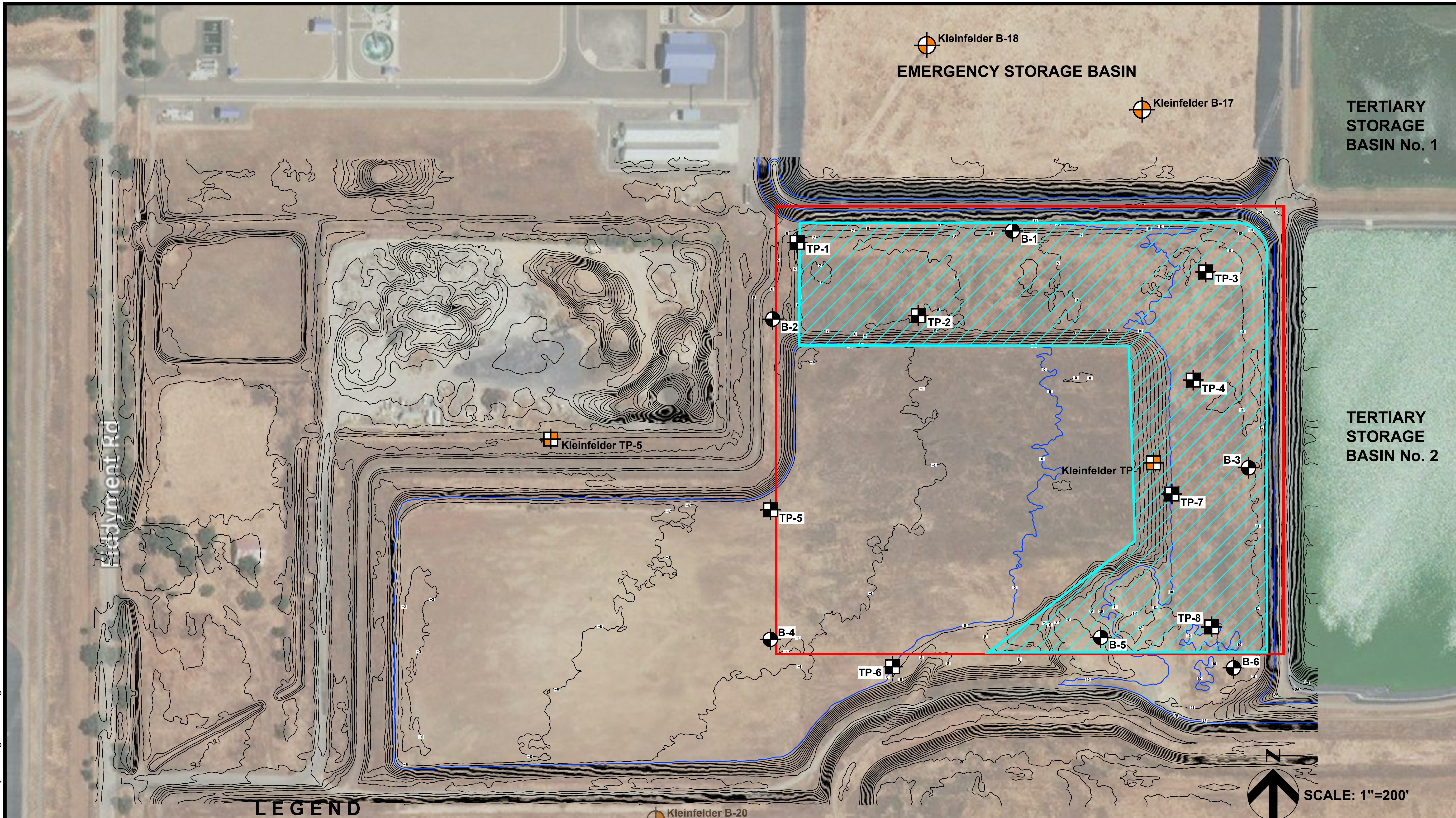
Lincoln Wastewater Treatment and Reclamation Facility Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, California

File No. 3228.x

April 2018

Figure 1

4/10/2018 3228.x Fig1 Third Tertiary Stoarage Basin.dwg



LEGEND

B-1 Approximate Boring Location

TP-1 Approximate Test Pit Location

Approx. Proposed Embankment Centerline

Approximate Previous Kleinfelder Exploration Locations

Approximate Proposed Borrow Area

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SITE MAP

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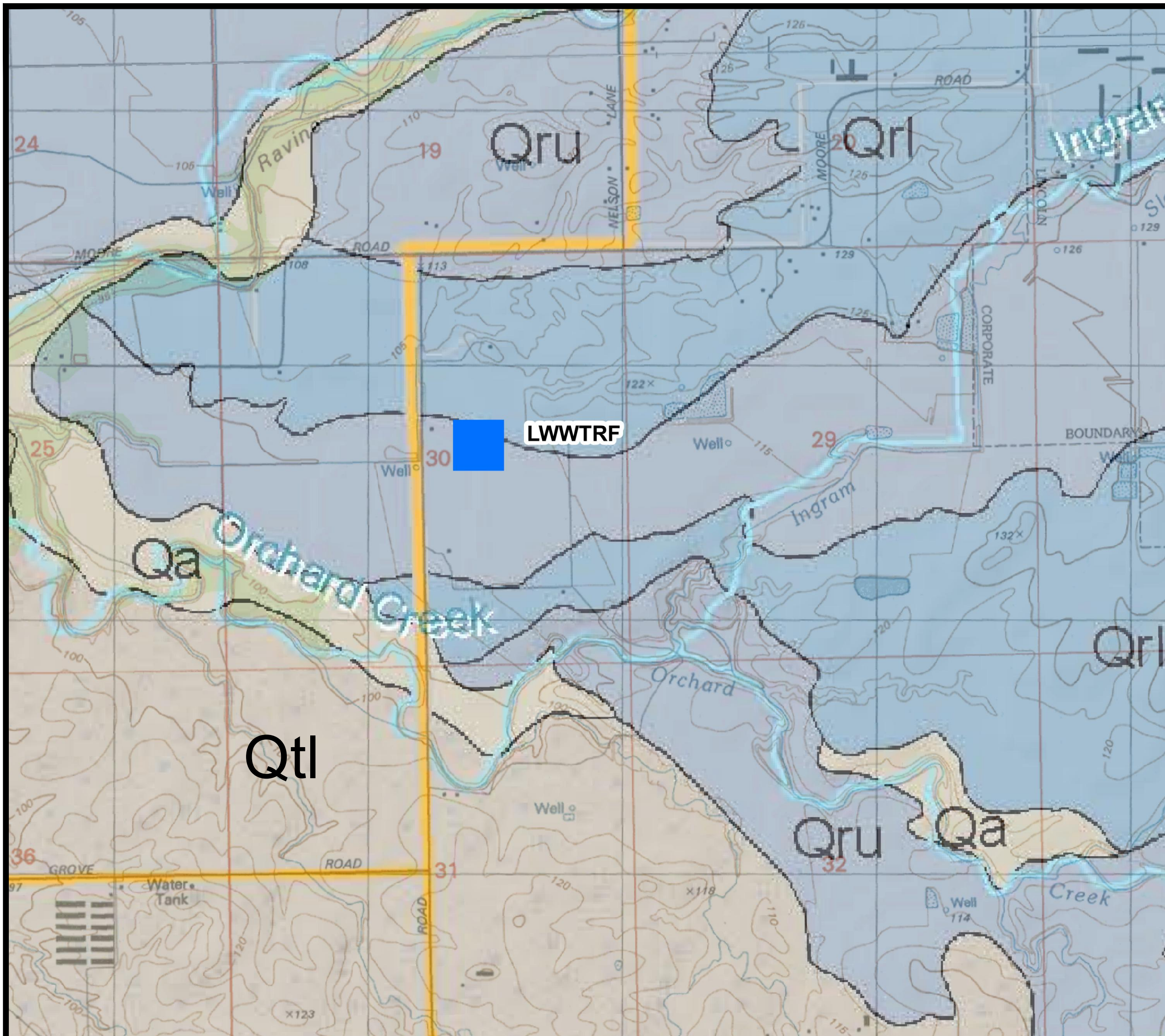
Tertiary Storage Basin No. 3
Placer County, California

File No. 3228.x

April 2018

Figure 2

4/10/2018 3228.x Fig2 Third Tertiary Storage Basin.dwg



LEGEND

Qa	Holocene alluvium- silt, sand, and gravel
Qb	Holocene basin deposits- fine grained silt and clay
Qru	Quaternary Upper Member, Riverbank Formation- unconsolidated silt, sand and gravel
Qrl	Quaternary Lower Member, Riverbank Formation- semiconsolidated silt, sand, and gravel
Qtl	Quaternary Turlock Lake Formation- silt, sand, and gravel

N

SCALE 1" = 2,000'

Source: MAPTECH Terrain Navigator Pro, v. 8.0, USGS topographic 7.5 minute quadrangle, Lincoln,1992, Pleasant Grove, 1967 (revised 1981),

Geologic Map of the Late Cenozoic Deposits of the Sacramento Valley and Northern Sierran Foothills, California, Helly, J.H., Hardwood, D.S., USGS, MF-1790, 1985, reproduced by State of California Department of Water Resources, 2006.

4/10/2018 3228.x Fig3 Third Tertiary Stoarage Basin.dwg

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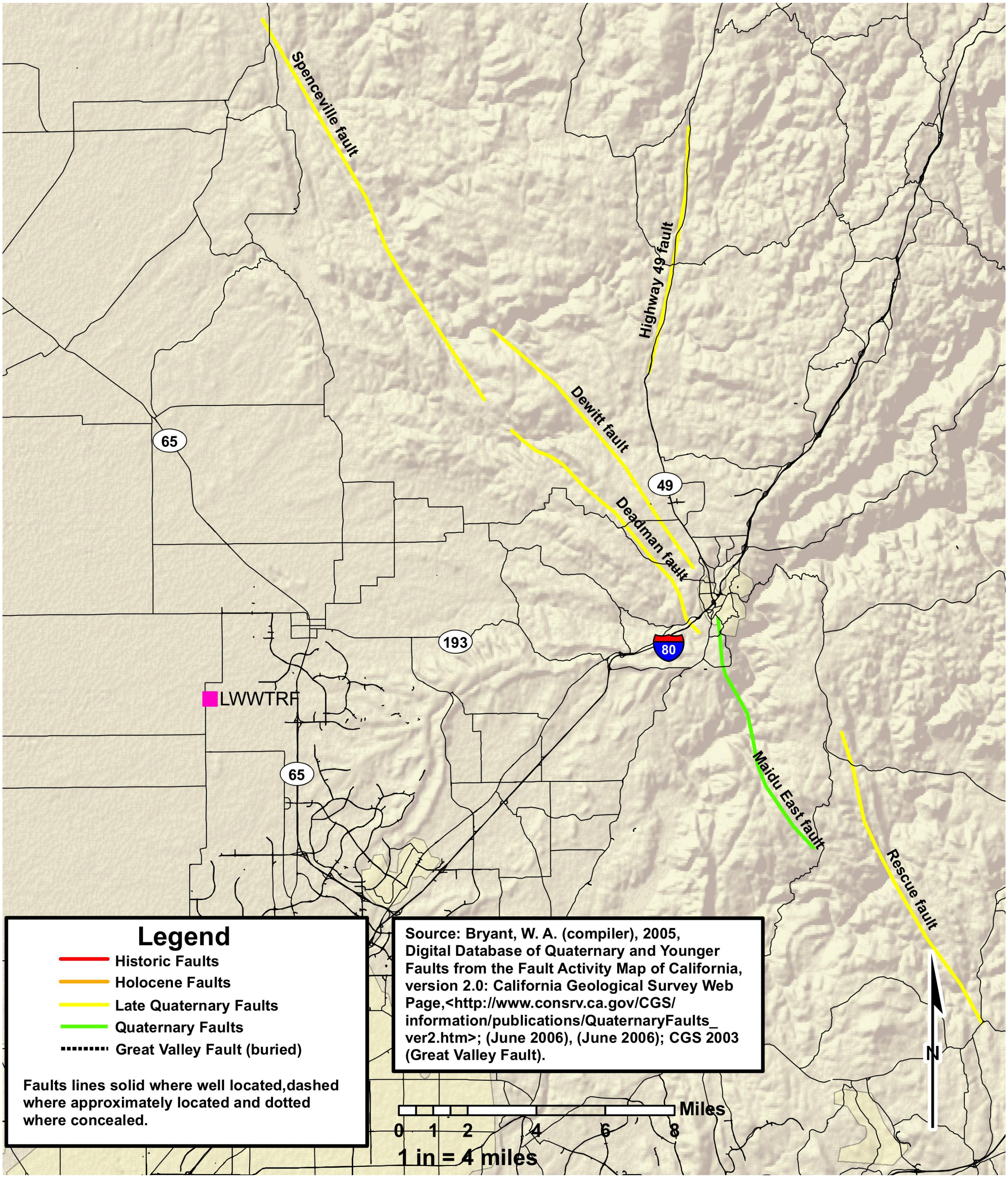
REGIONAL GEOLOGIC MAP

Lincoln Wastewater Treatment and Reclamation Facility Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, California

File No. 3228.x

April 2018

Figure 3



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REGIONAL FAULT MAP

Lincoln Wastewater Treatment and Reclamation Facility Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, California

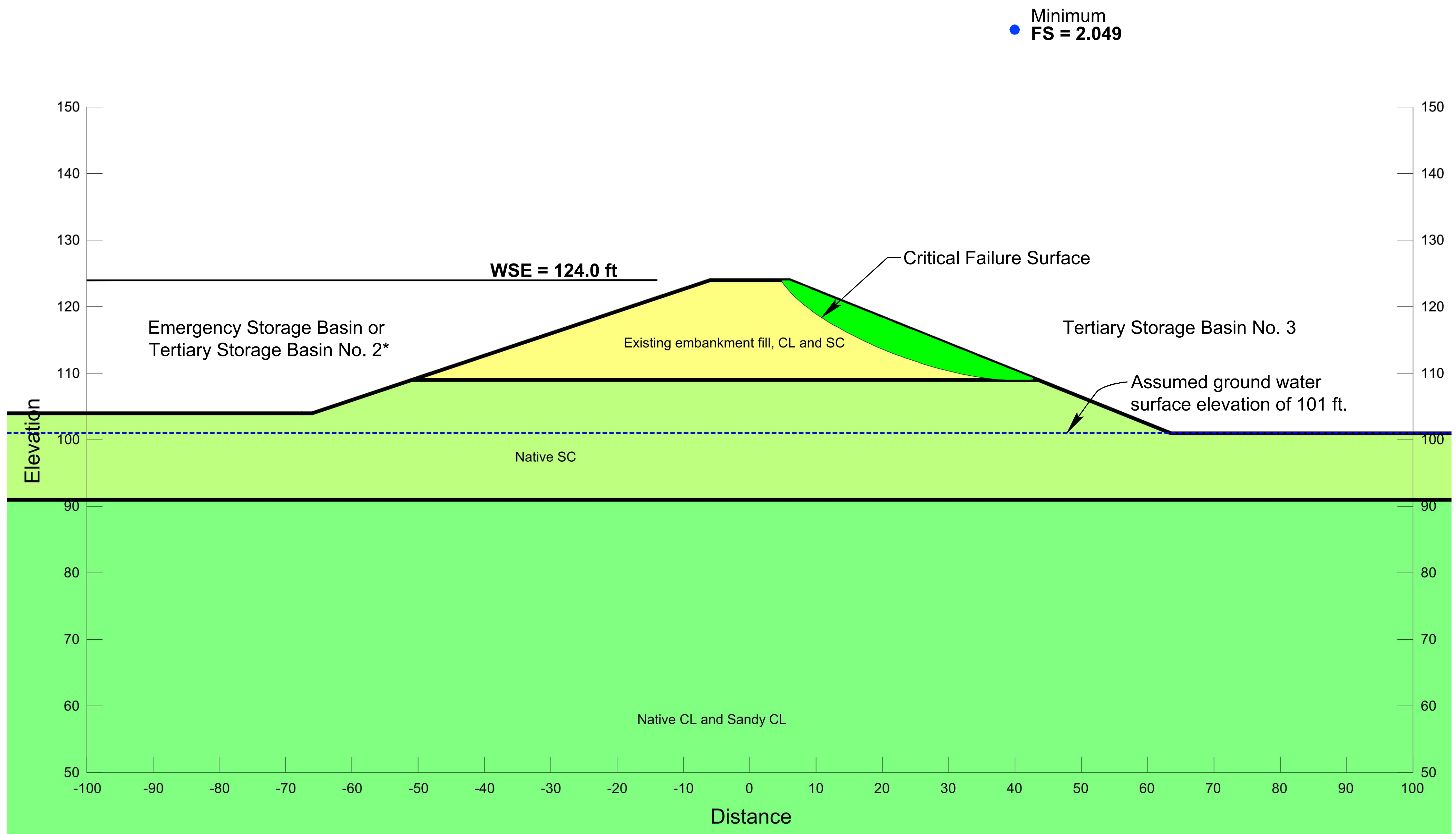
File No. 3228.x

April 2018

Figure 4

4/10/2018 3228.x Fig4 Third Tertiary Stoarage Basin.dwg

ANALYSIS OF AS DESIGNED EMBANKMENT
(Emergency Storage Basin/Tertiary Storage Basin No. 2 Full, Tertiary Basin No. 3 Empty)



* The Emergency Storage Basin, Tertiary Storage Basin No. 2, and Tertiary Storage Basin No. 3 are all lined with an HDPE liner.

North and East Embankment Cross-section			
Soil Description	ϕ'	c', psf	Unit weight, γ, pcf
Existing embankment fill, sandy lean clay, and clayey sands to elev. 109 ft	32°	50	129
Native clayey sands, elev. 91 to 109 ft	35°	110	126
Native sandy clays and lean clays below elev. 91 ft	0°	2000	122

SCALE: 1"=20' (approximate)

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NORTH AND EAST EMBANKMENT CROSS-SECTION, INNER SLOPE
Lincoln Wastewater Treatment and Reclamation Facility Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, California

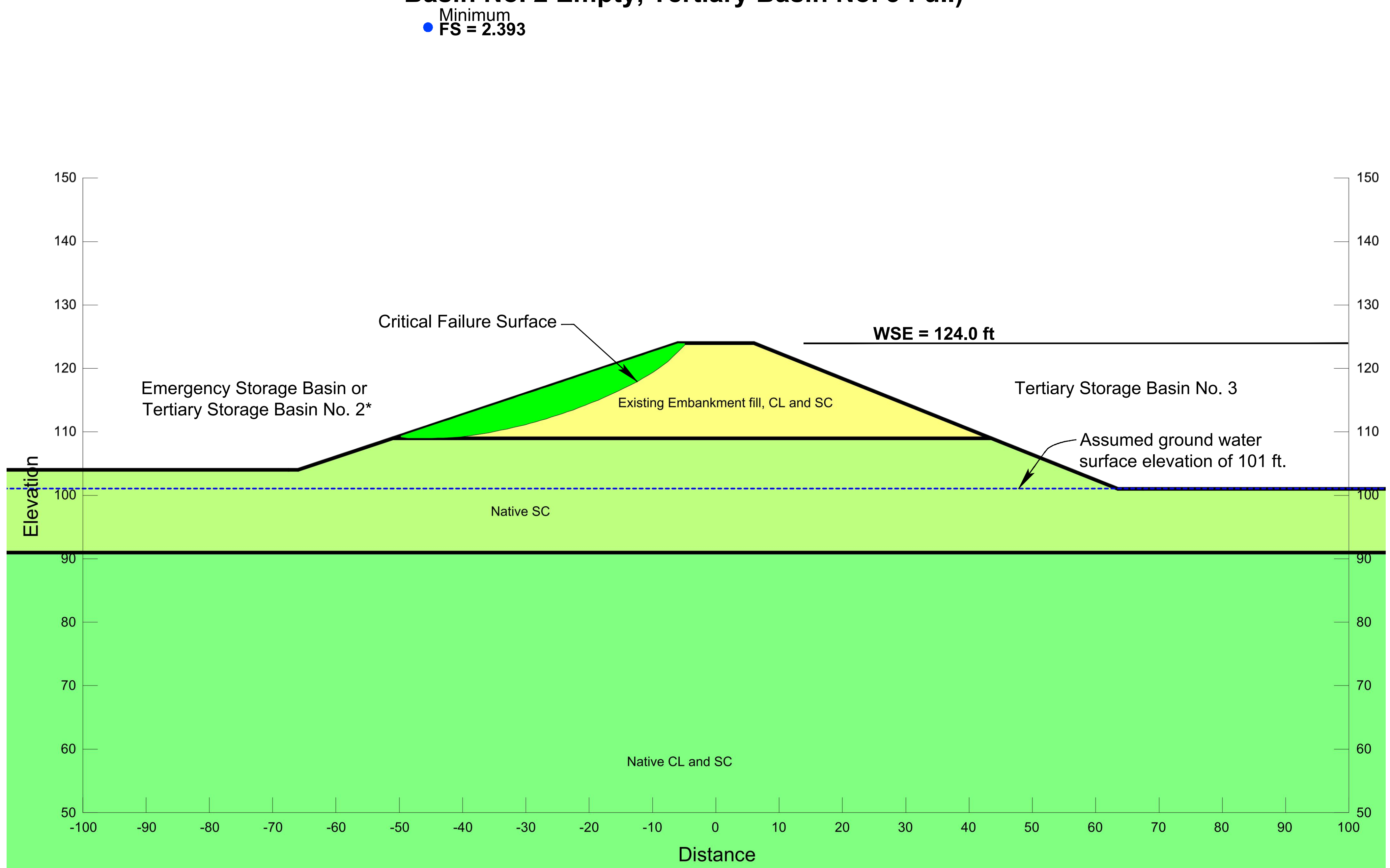
File No. 3228.x

April 2018

Figure 5

4/10/2018 3228.x Fig5 Third Tertiary Stoarage Basin.dwg

ANALYSIS OF AS DESIGNED EMBANKMENT
(Emergency Storage Basin/Tertiary Storage Basin No. 2 Empty, Tertiary Basin No. 3 Full)



North and East Embankment Cross-section			
Soil Description	ϕ'	c', psf	Unit weight, γ, pcf
Existing embankment fill, sandy lean clay, and clayey sands to elev. 109 ft	32°	50	129
Native clayey sands, elev. 91 to 109 ft	35°	110	126
Native sandy clays and lean clays below elev. 91 ft	0°	2000	122

* The Emergency Storage Basin, Tertiary Storage Basin No. 2, and Tertiary Storage Basin No. 3 are all lined with an HDPE liner.

SCALE: 1"=20' (approximate)

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NORTH AND EAST EMBANKMENT CROSS-SECTION, OUTER SLOPE
Lincoln Wastewater Treatment and Reclamation Facility Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, California

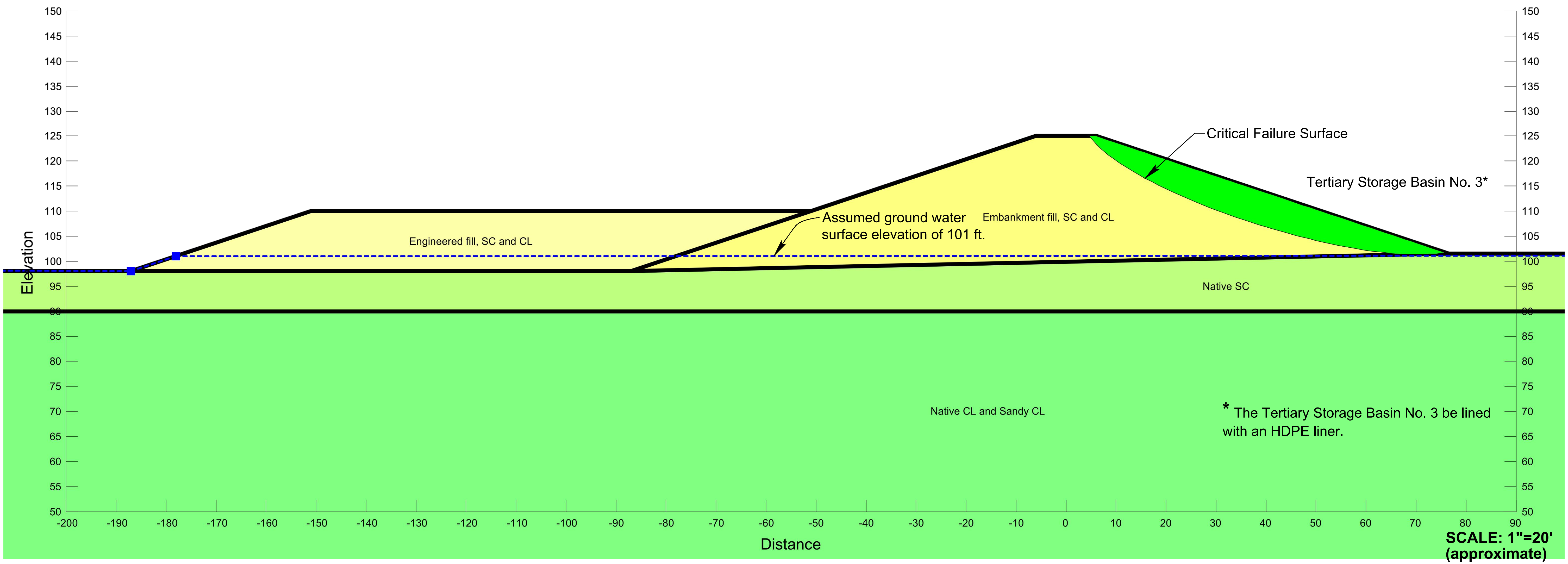
File No. 3228.x

April 2018

Figure 6

ANALYSIS OF AS DESIGNED EMBANKMENT
(South and West Embankment, Tertiary Basin No. 3 Empty)

South and West Embankment Cross-section			
Soil Description	**ϕ'**	**c', psf**	**Unit weight, γ, pcf**
Embankment fill: sandy lean clay and clayey sands to elev. 98 to 101.5 ft	32°	50	129
Engineered fill: sandy lean clay and clayey sands, elev. 98 to 110 ft	31°	25	129
Native clayey sands, elev. 90 to 101.5 ft	35°	110	126
Native sandy clays and lean clays below elev. 90 ft	0°	2000	122



SCALE: 1"=20' (approximate)

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SOUTH AND WEST EMBANKMENT CROSS-SECTION, INNER SLOPE
Lincoln Wastewater Treatment and Reclamation Facility Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, California

File No. 3228.x

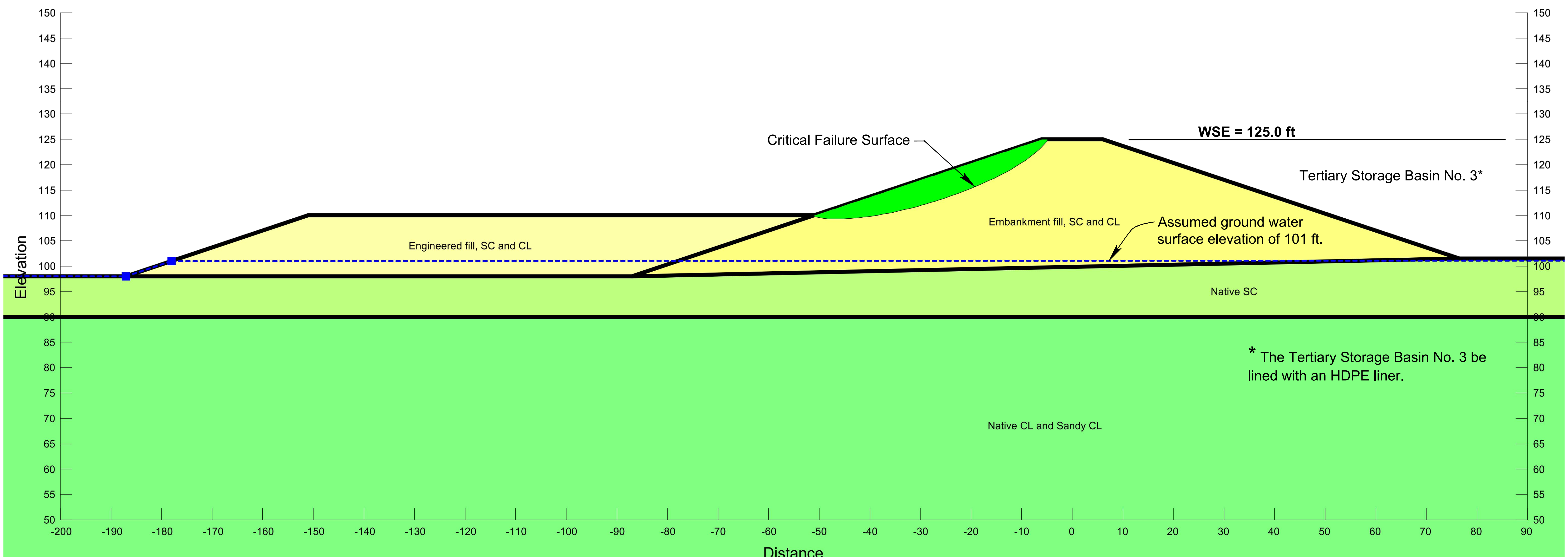
April 2018

Figure 7

4/10/2018 3228.x Fig7 Third Tertiary Stoarage Basin.dwg

ANALYSIS OF AS DESIGNED EMBANKMENT
(South and West Embankment, Tertiary Basin No. 3 Full)

South and West Embankment Cross-section			
Soil Description	ϕ'	c', psf	Unit weight, γ, pcf
Embankment fill: sandy lean clay and clayey sands to elev. 98 to 101.5 ft	32°	50	129
Engineered fill: sandy lean clay and clayey sands, elev. 98 to 110 ft	31°	25	129
Native clayey sands, elev. 90 to 101.5 ft	35°	110	126
Native sandy clays and lean clays below elev. 90 ft	0°	2000	122



SCALE: 1"=20' (approximate)

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SOUTH AND WEST EMBANKMENT CROSS-SECTION, OUTER SLOPE
Lincoln Wastewater Treatment and Reclamation Facility Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, California

File No. 3228.x

April 2018

Figure 8

4/10/2018 3228.x Fig8 Third Tertiary Stoarage Basin.dwg

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility

Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, CA

APPENDIX A

Boring Logs (B-1 through 6)
Legend to Logs
Test Pit Logs (TP-1 through 8)

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RMS	10-6-17	10-6-17		B1

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Taber		108.0 ft
DRILLING METHOD	**DRILL RIG**	**BOREHOLE DIAMETER**
Solid-Stem Auger	Diedrich D120	4 in
SAMPLER TYPE(S) AND SIZE(S) (ID)	**HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
2.4" CAMOD	Safety semi-automatic drop (140#/ 30")	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Backfill with Tremie Grout		None	None	26.5 ft

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	0		SANDY Lean CLAY with GRAVEL (CL), Hard, Light Brown, Dry, Fine SAND														
106.00	2				1	7 10 9	19	13	120					PP = >4.5			
104.00	3		CLAYEY SAND (SC); Medium Dense; Brown; Moist; Fine SAND														
102.00	5		Lean CLAY (CL); Very Stiff, Brown, Moist, Medium Cementation; Medium Plasticity		2	9 26 50/3"	76/9	19	105		14		UU = 2294.4	PP = 2.3	PI		
100.00	8		CLAYEY SAND (SC); Medium Dense; Reddish Brown; Moist; Fine SAND														
98.00	10				3	9 14 13	27										
92.00	15		SAND is Fine to Coarse; Some GRAVEL		4	5 7 15	22	14	119					PP = >4.5			
90.00	18		Lean CLAY (CL); Very Stiff; Brown; Moist; Medium Cementation; Low Plasticity; Traces of Fine SAND														
88.00	20				5	7 12 14	26							PP = 3.8			
82.00	25				6	5 8 12	20							PP = 3.6			

Bottom of borehole at 26.5 ft bgs

Backfill with Tremie Grout
No Groundwater Encountered
Bulk A: 0-5 ft
Bulk B: 5-10 ft

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PROJECT NAME		FILE NO.	HOLE ID
Lincoln WWTRF TSB No. 3		3228.X	B1
COUNTY	ROUTE	POSTMILE	
PLA		D	
CLIENT			
Stantec			
PREPARED BY	CHECKED BY	SHEET	
RMS	JTF	1 of 1	

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RMS	**10-6-17**	**10-6-17**		**B2**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Taber		**107.5 ft**

DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Solid-Stem Auger	**Diedrich D120**	**4 in**

SAMPLER TYPE(S) AND SIZE(S) (ID)	HAMMER TYPE	HAMMER EFFICIENCY, ERi
2.4" CAMOD	**Safety semi-automatic drop (140#/ 30")**	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Backfill with Tremie Grout		**15.0 ft**	**15.0 ft on14:00**	**26.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	0		Lean CLAY with SAND (CL); Hard; Light Brown; Dry; Fine SAND; Medium Cementation														
	1																
105.50	2				1	10 17 16	33							PP = >4.5			
	3																
103.50	4																
	5				2	8 4 17	21	13	119		28			PP = >4.5	PI		
101.50	6		Brown, Strong Cementation														
	7																
99.50	8																
	9																
97.50	10				3	6 9 13	22										
	11		CLAYEY SAND (SC); Light Brown														
			Poorly Graded SAND (SP); Medium Dense; Reddish Brown; Moist; Fine to Coarse SAND														
95.50	12																
	13																
93.50	14		CLAYEY SAND with GRAVEL (SC); Dark Yellowish Brown, Wet, Fine GRAVEL														
	15				4	9 12 16	28			21					PA		
91.50	16																
	17																
89.50	18		Lean CLAY (CL); Hard; Light Reddish Brown; Moist; Medium Cementation														
	19																
87.50	20				5	8 11 12	23	25	100					PP = 4.0			
	21																
85.50	22																
	23																
83.50	24																
	25		Light Brown, Traces of Fine SAND		6	9 11 10	21							PP = 4.0			
81.50	26																
	27		Bottom of borehole at 26.5 ft bgs														
79.50	28		Backfill with Tremie Grout Groundwater at 15 ft Bulk A: 0-5 ft Bulk B: 5-10 ft														
	29																
	30																

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PROJECT NAME		FILE NO.	HOLE ID
Lincoln WWTRF TSB No. 3		**3228.X**	**B2**
COUNTY **PLA**	ROUTE	POSTMILE **D**	
CLIENT **Stantec**			
PREPARED BY **RMS**	CHECKED BY **JTF**	SHEET **1 of 1**	

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RMS	10-6-17	10-6-17		B3

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Taber		111.0 ft
DRILLING METHOD	**DRILL RIG**	**BOREHOLE DIAMETER**
Solid-Stem Auger	Diedrich D120	4 in
SAMPLER TYPE(S) AND SIZE(S) (ID)	**HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
2.4" CAMOD	Safety semi-automatic drop (140#/ 30")	
BOREHOLE BACKFILL AND COMPLETION	**GROUNDWATER READINGS** — DURING DRILLING: None; AFTER DRILLING (DATE): None	**TOTAL DEPTH OF BORING**
Backfill with Tremie Grout		26.5 ft

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION/REMARKS	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests
111.00	0	CLAYEY SAND (SC); Medium Dense; Reddish Brown; Moist; Fine SAND											
109.00	2		1	11 9 8	17								
107.00	4												
	5		2	7 14 14	28	6	105			39.1	DS = 115.8		DS
105.00	6	Poorly Graded SAND with CLAY (SP-SC); Medium Dense; Strong Brown; Moist; Fine to Coarse SAND											
103.00	8	SANDY Lean CLAY (CL); Hard; Reddish Brown; Moist; Fine to Coarse SAND											
101.00	10		3	11 14 16	30	11	110					PP = >4.5	
97.00	14	CLAYEY SAND (SC); Medium Dense; Olive Brown; Fine SAND											
	15		4	6 10 9	19							PP = 3.6	
95.00	16	Lean CLAY (CL); Very Stiff to Hard; Light Olive Brown; Moist; Medium Plasticity											
93.00	18	Reddish Brown											
91.00	20		5	7 10 12	22	24	102				UU = 2337.4	PP = 4.2	
	23	Stiff; Light Olive Brown											
	25		6	4 4 5	9							PP = 2.5	
85.00	26.5	Bottom of borehole at 26.5 ft bgs											

Backfill with Tremie Grout
No Groundwater Encountered
Bulk A: 0-5 ft
Bulk B: 5-10 ft

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PROJECT NAME		FILE NO.	HOLE ID
Lincoln WWTRF TSB No. 3		3228.X	B3
COUNTY: PLA	ROUTE:	POSTMILE: D	
CLIENT: Stantec			
PREPARED BY: RMS	CHECKED BY: JTF	SHEET: 1 of 1	

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RMS	10-6-17	10-6-17		B4

DRILLING CONTRACTOR Taber	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION 99.0 ft
DRILLING METHOD Solid-Stem Auger	DRILL RIG Diedrich D120	BOREHOLE DIAMETER 4 in
SAMPLER TYPE(S) AND SIZE(S) (ID) 2.4" CAMOD	HAMMER TYPE Safety semi-automatic drop (140#/ 30")	HAMMER EFFICIENCY, ERi
BOREHOLE BACKFILL AND COMPLETION Backfill with Tremie Grout	GROUNDWATER READINGS — DURING DRILLING: 18.0 ft; AFTER DRILLING (DATE): 18.0 ft on12:20	TOTAL DEPTH OF BORING 26.5 ft

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION/REMARKS	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests
	0	CLAYEY SAND (SC); Medium Dense; Brown; Dry; Fine SAND											
97.00	2		1	4 10 12	22	12	109					PP = >4.5	
	3	SANDY Lean CLAY (CL); Hard; Reddish Brown; Moist; Fine to Coarse SAND											
	5	CLAYEY SAND (SC); Very Dense; Olive Brown; Moist; Fine to Medium SAND	2	22 41 38	79							PP = >4.5	
91.00	8	Lean CLAY (CL); Hard; Brown; Moist; Traces of Fine SAND; Medium Plasticity											
89.00	10	Very Stiff; Yellowish Brown; Weak Cementation; Low Plasticity	3	7 14 16	30	22	107					PP = >4.5	
	15	No Cementation	4	6 12 14	26							PP = 4.2	
81.00	18	(Groundwater)											
79.00	20	Stiff; Light Olive Brown; Medium Plasticity	5	3 5 6	11							PP = 1.3	
	25	Hard	6	7 12 15	27							PP = 4.5	

Elevation scale: 97.00 (2), 95.00 (4), 93.00 (6), 91.00 (8), 89.00 (10), 87.00 (12), 85.00 (14), 83.00 (16), 81.00 (18), 79.00 (20), 77.00 (22), 75.00 (24), 73.00 (26), 71.00 (28); depth 0–30.

Bottom of borehole at 26.5 ft bgs

Backfill with Tremie Grout
Groundwater at 18 ft
Bulk A: 0-5 ft
Bulk B: 5-10 ft

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PROJECT NAME	FILE NO.	HOLE ID
Lincoln WWTRF TSB No. 3	3228.X	B4

COUNTY	ROUTE	POSTMILE
PLA		D

CLIENT		
Stantec		

PREPARED BY	CHECKED BY	SHEET
RMS	JTF	1 of 1

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RMS	10-6-17	10-6-17		B5

DRILLING CONTRACTOR **Taber**	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION **108.0 ft**
DRILLING METHOD **Solid-Stem Auger**	DRILL RIG **Diedrich D120**	BOREHOLE DIAMETER **4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID) **2.4" CAMOD**	HAMMER TYPE **Safety semi-automatic drop (140#/ 30")**	HAMMER EFFICIENCY, ERi
BOREHOLE BACKFILL AND COMPLETION **Backfill with Tremie Grout**	GROUNDWATER READINGS — DURING DRILLING **None** — AFTER DRILLING (DATE) **None**	TOTAL DEPTH OF BORING **26.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	0		SANDY Lean CLAY (CL); Hard; Light Brown; Dry; Fine SAND; Medium Cementation														
	1																
106.00	2				1	14 11 10	21	11	99					PP = >4.5			
	3																
104.00	4																
	5		Yellowish Brown; Very Stiff; Moist; Medium to Strong Cementation		2	7 7 6	13	19	98				UU = 2728.4	PP = 3.5			
102.00	6																
	7																
100.00	8		Lean CLAY (CL); Hard; Light Yellowish Brown; Moist; Medium Cementation														
	9																
98.00	10				3	12 33 35	68	34	88				UU = 6022.4	PP = >4.5			
	11																
96.00	12																
	13																
94.00	14																
	15				4	9 18 22	40	40	79					PP = >4.5			
92.00	16																
	17																
90.00	18																
	19																
88.00	20		Light Reddish Brown, Traces of Fine SAND		5	5 13 17	30							PP = >4.5			
	21																
86.00	22																
	23																
84.00	24																
	25		Light Brown; Weak Cementation		6	5 7 13	20							PP = >4.5			
82.00	26																
	27		Bottom of borehole at 26.5 ft bgs														
80.00	28		Backfill with Tremie Grout No Groundwater Encountered Bulk A: 0-5 ft Bulk B: 5-10 ft														
	29																
	30																

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PROJECT NAME		FILE NO.	HOLE ID
Lincoln WWTRF TSB No. 3		3228.X	B5
COUNTY PLA	ROUTE	POSTMILE D	
CLIENT Stantec			
PREPARED BY RMS	CHECKED BY JTF	SHEET 1 of 1	

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RMS	**10-6-17**	**10-6-17**		**B6**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Taber		**110.0 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Solid-Stem Auger	**Diedrich D120**	**4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID)	HAMMER TYPE	HAMMER EFFICIENCY, ERi
2.4" CAMOD	**Safety semi-automatic drop (140#/ 30")**	
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **None**; AFTER DRILLING (DATE): **None**	TOTAL DEPTH OF BORING
Backfill with Tremie Grout		**26.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	0–1		Lean CLAY with SAND (CL); Very Stiff; Reddish Brown; Moist; Fine to Medium SAND; Low to Medium Plasticity														
108.00	2				1	7 8 10	18							PP = 3.5			
106.00	4																
	4.5		SANDY Lean CLAY (CL); Hard; Reddish Brown; Moist; Fine to Coarse SAND; Low Plasticity		2	9 16 18	34	17	117					PP = >4.5			
102.00	8		CLAYEY SAND (SC); Medium Dense; Yellowish Brown; Moist; Medium to Fine SAND														
100.00	10				3	5 10 14	24	25	97	37				PP = >4.5	PA		
	13		SILT (ML); Hard; Light Brown; Moist; Weak Cementation														
	15				4	7 15 16	31	31	91					PP = >4.5			
92.00	18		Lean CLAY (CL); Very Stiff; Light Brown; Moist; Weak Cementation: Low Plasticity														
90.00	20				5	8 17 17	34							PP = 3.75			
	25		Light Olive Brown		6	10 16 20	36							PP = 3.75			

Elevations: 108.00, 106.00, 104.00, 102.00, 100.00, 98.00, 96.00, 94.00, 92.00, 90.00, 88.00, 86.00, 84.00, 82.00 (depths 2–28 ft)

Bottom of borehole at 26.5 ft bgs

Backfill with Tremie Grout
No Groundwater Encountered
Bulk A: 0-5 ft
Bulk B: 5-10 ft

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PROJECT NAME		FILE NO.	HOLE ID
Lincoln WWTRF TSB No. 3		**3228.X**	**B6**
COUNTY: **PLA**	ROUTE	POSTMILE: **D**	
CLIENT: **Stantec**			
PREPARED BY: **RMS**	CHECKED BY: **JTF**	SHEET: **1 of 1**	

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (CTM 216 - 06)
CR	Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content (ASTM D 2974-07)
P	Permeability (CTM 220 - 05)
PA	Particle Size Analysis (ASTM D 422-63 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301 - 00)
SE	Sand Equivalent (CTM 217 - 99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96 [2004])

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- 2.5" ID Sampler
- 2" ID Sampler
- Shelby Tube
- Piston Sampler
- NX Rock Core
- HQ Rock Core
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (short-term)
- Static Water Level Reading (long-term)

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BORING RECORD LEGEND

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60} - Value (blows / foot)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Descriptor	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Descriptor	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE: This legend sheet provides descriptors and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010), Section 2, for tables of additional soil description components and discussion of soil description and identification.

TEST PIT LOG

Test Pit No: TP1
Sheet 1 of 1

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West Sacramento 95691
Telephone: 916 375 8706
Fax: 916 375 8709

Project No.:	3228.X
Project Name:	Lincoln WWTRF
Project Location:	Lincoln, CA
Logged By:	RMS Date: 10/31/2017
Contractor:	Lic. No.
Operator:	Rob Rasch
Backhoe Type:	Bobcat E32
Ground Elevation:	107 ft Depth: 9 ft

Sketch

Ground Water Elevation Data

Date Time Depth	No Groundwater Encountered

	Pocket Pen (tsf)	Blow Counts	Depth in (ft)	Sample Interval & No	Graphic Log	Description
			1			SANDY Lean CLAY (CL); Dry; Light Brown
			2			
			3			Moist; Brown; Medium Plasticity
			4			
			5			CLAYEY SAND (SC); Moist; Reddish Brown; Low Plasticity; Fine SAND
			6			
			7			
			8			
			9			
			10			End of Boring at 9 ft Bulk A: 0-4ft Bulk B: 4-9ft
			11			
			12			
			13			
			14			
			15			
			16			

TEST PIT LOG

		Test Pit No:	TP2
blackburn consulting 2491 Boatman Ave West Sacramento 95691 Telephone: 916 375 8706 Fax: 916 375 8709	Project No.: 3228.X	Sheet 1 of 1	
	Project Name: Lincoln WWTRF		
	Project Location: Lincoln, CA		
	Logged By: RMS Date: 10/31/2017		
Sketch	Contractor:	Lic. No.	
	Operator: Rob Rasch		
	Backhoe Type: Bobcat E32		
	Ground Elevation: 108 ft Depth: 8 ft		

	Pocket Pen (tsf)	Blow Counts	Depth in (ft)	Sample Interval & No	Graphic Log	Description	Ground Water Elevation Data: Date Time Depth	No Groundwater Encountered
			1			SANDY Lean CLAY (CL); Medium Stiff; Reddish Brown; Dry; Fine SAND; Low Plasticity		
			2					
			3			Brown; Moist		
			4					
			5			Dry; Light Brown; Medium Cementation		
			6					
			7			Brown; Moist		
			8					
			9			End of Boring at 8 ft Bulk A: 0-4ft Bulk B: 4-8ft		
			10					
			11					
			12					
			13					
			14					
			15					
			16					

TEST PIT LOG

Test Pit No:	TP3
Sheet 1 of	1

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2491 Boatman Ave
West Sacramento 95691
Telephone: 916 375 8706
Fax: 916 375 8709

Project No.:	3228.X
Project Name:	Lincoln WWTRF
Project Location:	Lincoln, CA
Logged By:	RMS Date: 10/31/2017
Contractor:	Lic. No.
Operator:	Rob Rasch
Backhoe Type:	Bobcat E32
Ground Elevation:	110.5 ft Depth: 8.5 ft

Sketch

Ground Water Elevation Data	
Date Time Depth	No Groundwater Encountered

	Pocket Pen (tsf)	Blow Counts	Depth in (ft)	Sample Interval & No	Graphic Log	Description
			1			SILT (ML); Light Brown; Dry; Fine SAND; Weak Cementation; PI=5 LL=31
			2			
			3			
			4			SANDY Lean CLAY (CL); Brown; Moist; Fine SAND
			5			
			6			
			7			
			8			Reddish Brown
			9			
			10			End of Boring at 8.5 ft Bulk A: 0-3ft Bulk B: 3-8.5ft
			11			
			12			
			13			
			14			
			15			
			16			

TEST PIT LOG

Test Pit No: TP4

Sheet 1 of 1

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West Sacramento 95691
Telephone: 916 375 8706
Fax: 916 375 8709

Project No.:	3228.X
Project Name:	Lincoln WWTRF
Project Location:	Lincoln, CA
Logged By:	RMS Date: 10/31/2017
Contractor:	Lic. No.
Operator:	Rob Rasch
Backhoe Type:	Bobcat E32
Ground Elevation:	110.5 ft Depth: 8.5 ft

Sketch

Ground Water Elevation Data

Date Time Depth	No Groundwater Encountered

	Pocket Pen (tsf)	Blow Counts	Depth in (ft)	Sample Interval & No	Graphic Log	Description
			1			
			2			
			3			Reddish Brown
			4			
			5			
			6			
			7			
			8			
			9			
			10			End of Boring at 8.5 ft Bulk A: 0-8.5ft
			11			
			12			
			13			
			14			
			15			
			16			

TEST PIT LOG

		Test Pit No:	TP5
Project No.:	3228.X	Sheet 1 of	1
Project Name:	Lincoln WWTRF		
Project Location:	Lincoln, CA		
Logged By:	RMS	Date:	10/31/2017
Contractor:		Lic. No.	
Operator:	Rob Rasch		
Backhoe Type:	Bobcat E32		
Ground Elevation:	99 ft	Depth:	8.5 ft

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Telephone: 916 375 8706
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Sketch

Ground Water Elevation Data

Date Time Depth	No Groundwater Encountered

	Pocket Pen (tsf)	Blow Counts	Depth in (ft)	Sample Interval & No	Graphic Log	Description
			1			Well Graded SAND with CLAY (SW-SC); Moist; Reddish Brown
			2			
			3			
			4			Lean CLAY (CL); Moist; Light Olive Brown; Low to Medium Plasticity; Traces of Fine SAND
			5			Reddish Brown
			6			CLAYEY SAND (SC); Moist; Light Reddish Brown; Fine SAND
			7			
			8			
			9			
			10			End of Boring at 8.5 ft Bulk A: 0-3ft Bulk B: 3-5ft Bulk C: 5-8.5ft
			11			
			12			
			13			
			14			
			15			
			16			

TEST PIT LOG

Test Pit No:	TP6
Sheet 1 of	1

blackburn consulting
2491 Boatman Ave
West Sacramento 95691
Telephone: 916 375 8706
Fax: 916 375 8709

Project No.:	3228.X		
Project Name:	Lincoln WWTRF		
Project Location:	Lincoln, CA		
Logged By:	RMS	Date:	10/31/2017
Contractor:		Lic. No.	
Operator:	Rob Rasch		
Backhoe Type:	Bobcat E32		
Ground Elevation:	100 ft	Depth:	6.5 ft

Sketch

Ground Water Elevation Data	
Date Time Depth	No Groundwater Encountered

	Pocket Pen (tsf)	Blow Counts	Depth in (ft)	Sample Interval & No	Graphic Log	Description
			1			CLAYEY SAND (SC); Dry; Light Brown; Fine SAND; Medium to Strong Cementation
			2			
			3			
			4			Strongly Cemented Clumps
			5			Light Beige
			6			
			7			
			8			End of Boring at 6.5 ft Backhoe Refusal Bulk A: 0-6.5ft
			9			
			10			
			11			
			12			
			13			
			14			
			15			
			16			

TEST PIT LOG

Test Pit No:	TP7
Sheet 1 of	1

blackburn consulting
2491 Boatman Ave
West Sacramento 95691
Telephone: 916 375 8706
Fax: 916 375 8709

Project No.:	3228.X
Project Name:	Lincoln WWTRF
Project Location:	Lincoln, CA
Logged By:	RMS Date: 10/31/2017
Contractor:	Lic. No.
Operator:	Rob Rasch
Backhoe Type:	Bobcat E32
Ground Elevation:	109 ft Depth: 9 ft

Sketch

Ground Water Elevation Data

Date Time Depth	No Groundwater Encountered

	Pocket Pen (tsf)	Blow Counts	Depth in (ft)	Sample Interval & No	Graphic Log	Description
			1			CLAYEY SAND (SC); Olive Brown; Moist
			2			
			3			
			4			CLAYEY SAND (SC); Brown; Moist
			5			
			6			
			7			
			8			Reddish Brown
			9			
			10			End of Boring at 9 ft Bulk A: 0-3ft Bulk B: 3-7ft Bulk C: 7-9ft
			11			
			12			
			13			
			14			
			15			
			16			

TEST PIT LOG

Test Pit No:	TP8		
Sheet 1 of 1			

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2491 Boatman Ave
West Sacramento 95691
Telephone: 916 375 8706
Fax: 916 375 8709

Project No.:	3228.X		
Project Name:	Lincoln WWTRF		
Project Location:	Lincoln, CA		
Logged By:	RMS	Date:	10/31/2017
Contractor:		Lic. No.	
Operator:	Rob Rasch		
Backhoe Type:	Bobcat E32		
Ground Elevation:	109.5 ft	Depth:	9 ft

Sketch

Ground Water Elevation Data	
Date Time Depth	No Groundwater Encountered

	Pocket Pen (tsf)	Blow Counts	Depth in (ft)	Sample Interval & No	Graphic Log	Description
			1			CLAYEY SAND (SC); Light Brown; Dry; Fine SAND
			2			
			3			
			4			CLAYEY SAND (SC); Reddish Brown; Moist; Traces of GRAVEL
			5			
			6			
			7			Well Graded SAND with CLAY (SW-SC); Reddish Brown; Moist
			8			
			9			
			10			End of Boring at 9 ft Bulk A: 0-3ft Bulk B: 3-6ft Bulk C: 6-9ft
			11			
			12			
			13			
			14			
			15			
			16			

GEOTECHNICAL DESIGN REPORT

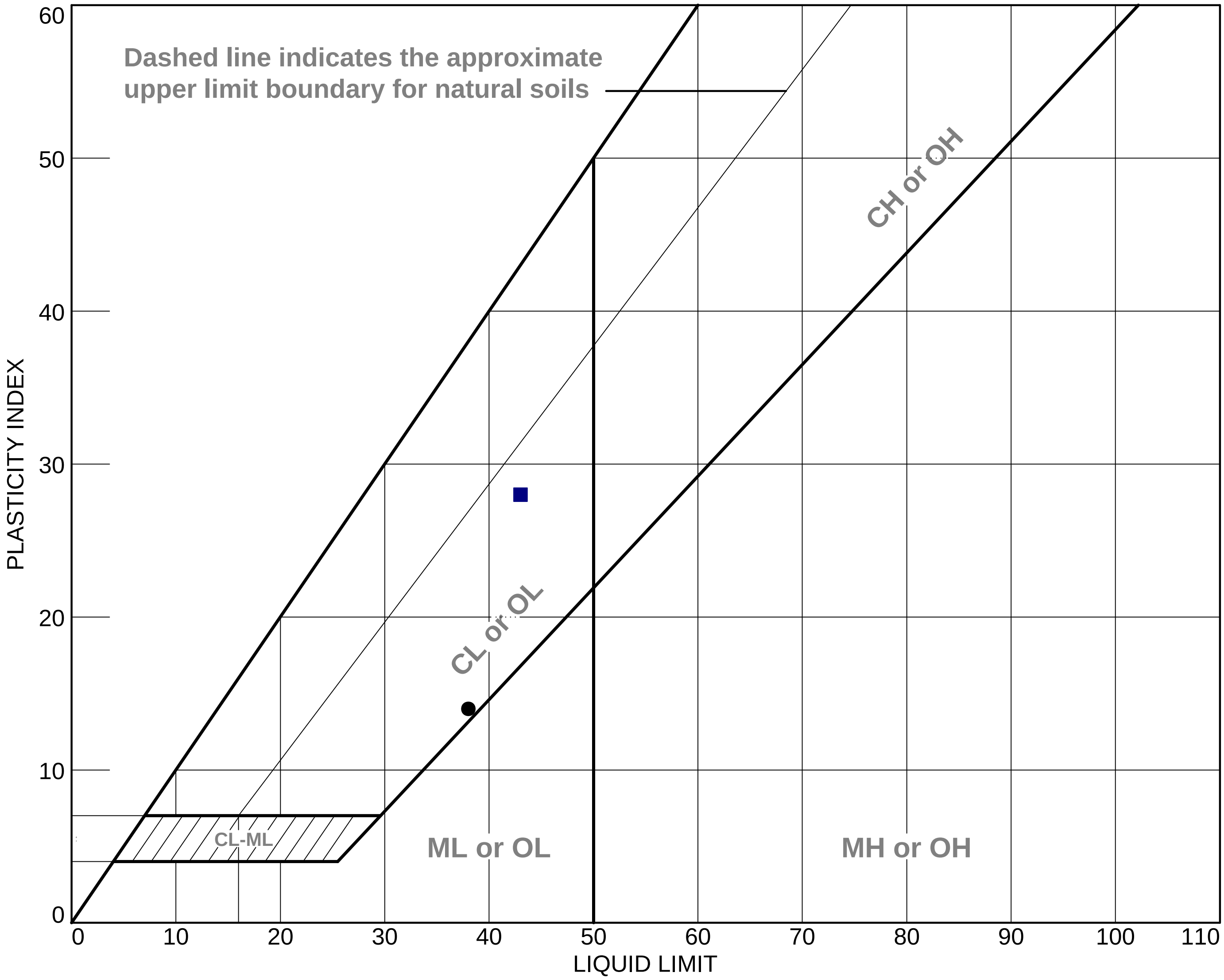
Lincoln Wastewater Treatment and Reclamation Facility

Phase 1 Expansion
Tertiary Storage Basin No. 3
Placer County, CA

APPENDIX B

Laboratory Summary
Laboratory Test Results

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B1	2C	5.75-6.25	---	24	38	14	CL
■	B2	2C	6.0-6.5'	---	15	43	28	CL

Blackburn Consulting

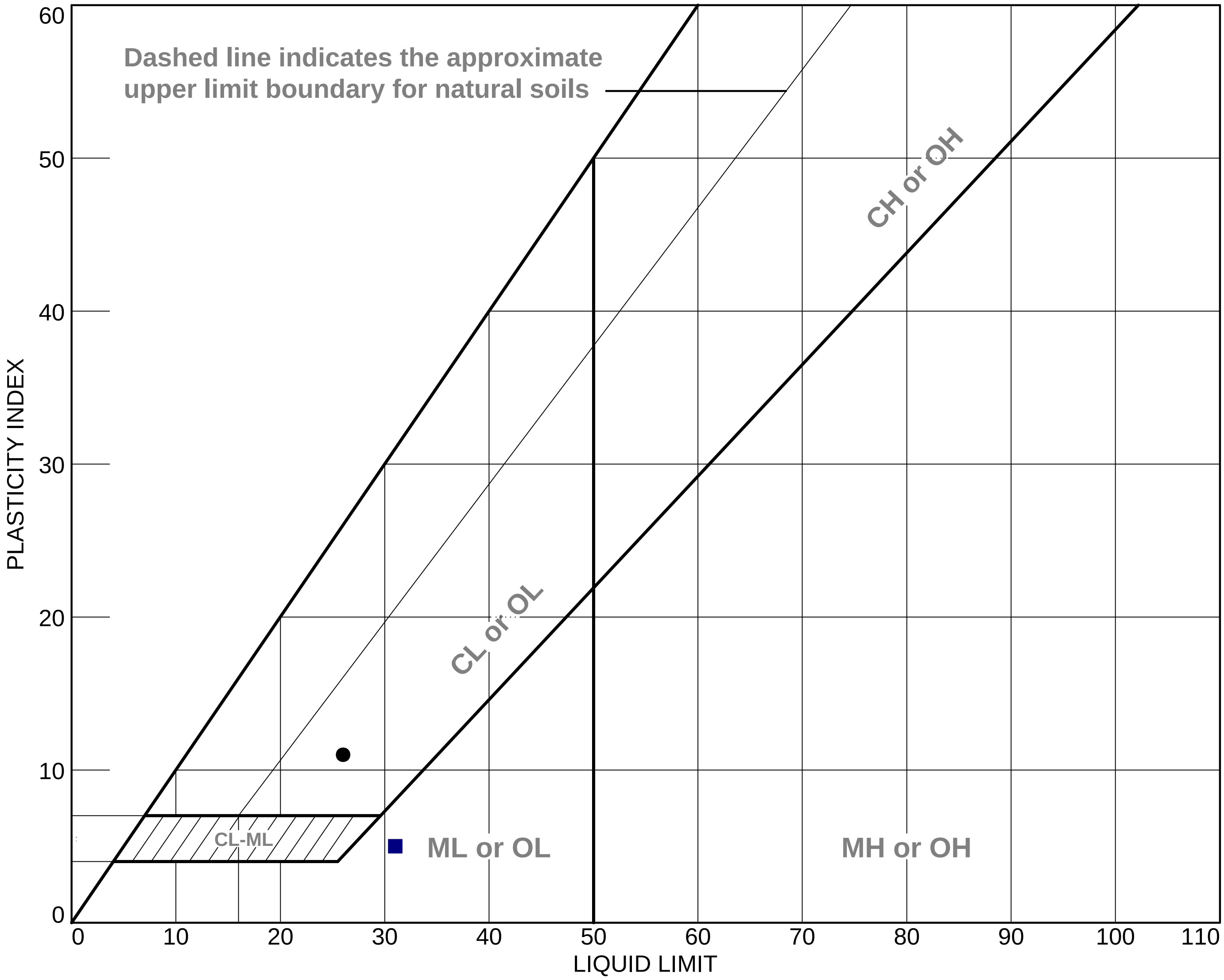
W. Sacramento, CA

Client: Stantec - Rocklin

Project: LWWTRF Expansion Phase 1&2

Project No.: 3228.X **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



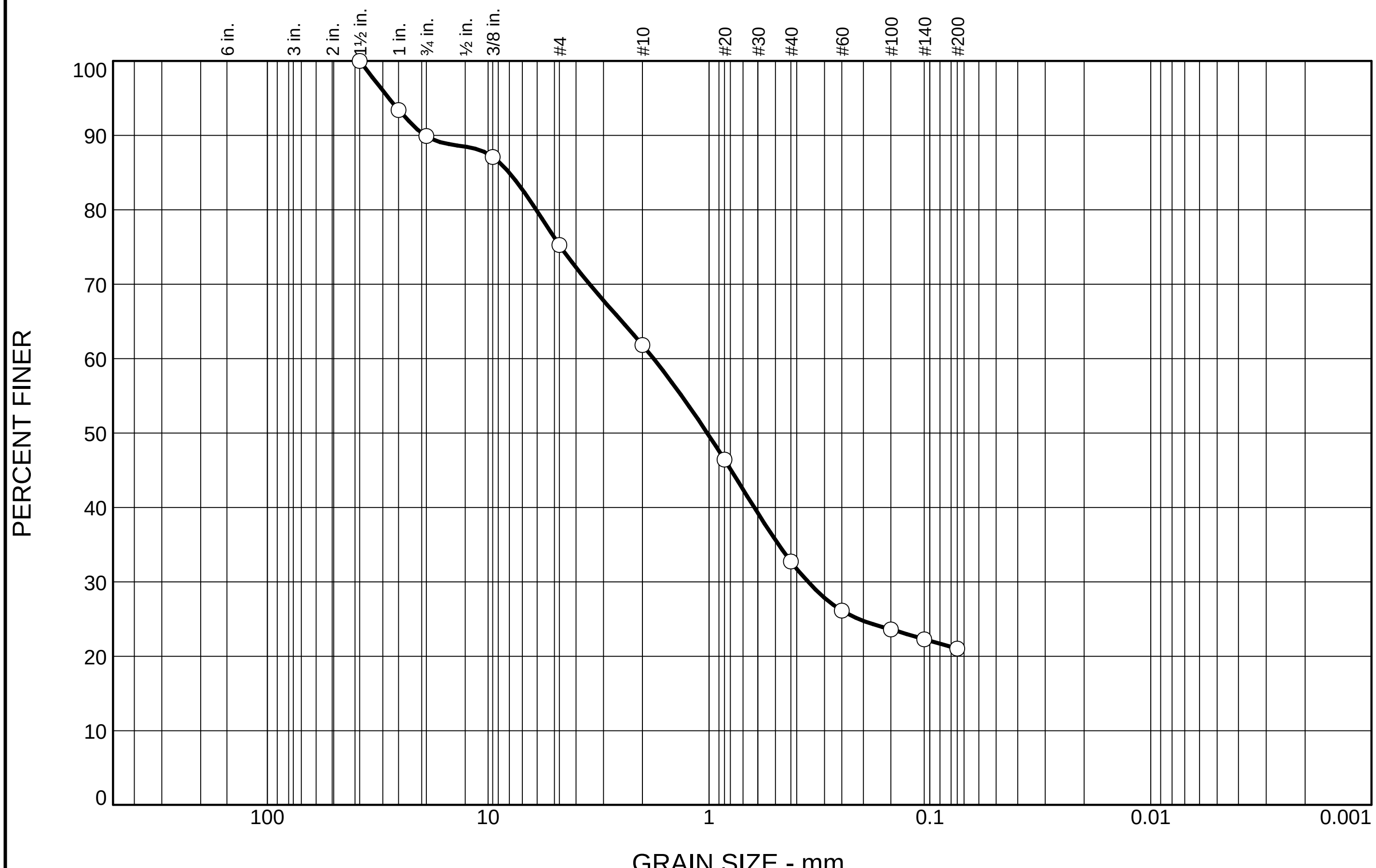
SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	TP1	Bulk A	0.0-4.0'	---	15	26	11	CL
■	TP3	Bulk A	0.0-3.0'	---	26	31	5	ML

Blackburn Consulting

W. Sacramento, CA

Client: Stantec - Rocklin
Project: LWWTRF Expansion Phase 1&2
Project No.: 3228.X **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	10	15	13	29	12	21	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100		
1"	93		
3/4"	90		
3/8"	87		
#4	75		
#10	62		
#20	46		
#40	33		
#60	26		
#100	24		
#140	22		
#200	21		

* (no specification provided)

Material Description

CLAYEY SAND with GRAVEL, strong brown

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 19.2499 D_{85}= 8.0267 D_{60}= 1.7835

D_{50}= 1.0184 D_{30}= 0.3560 D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

ASTM D6913 mass reqs. not met due to >1" gravel in sample

Source of Sample: B2 **Depth:** 16.0-16.5'
Sample Number: 4C

Date: 11/20/17

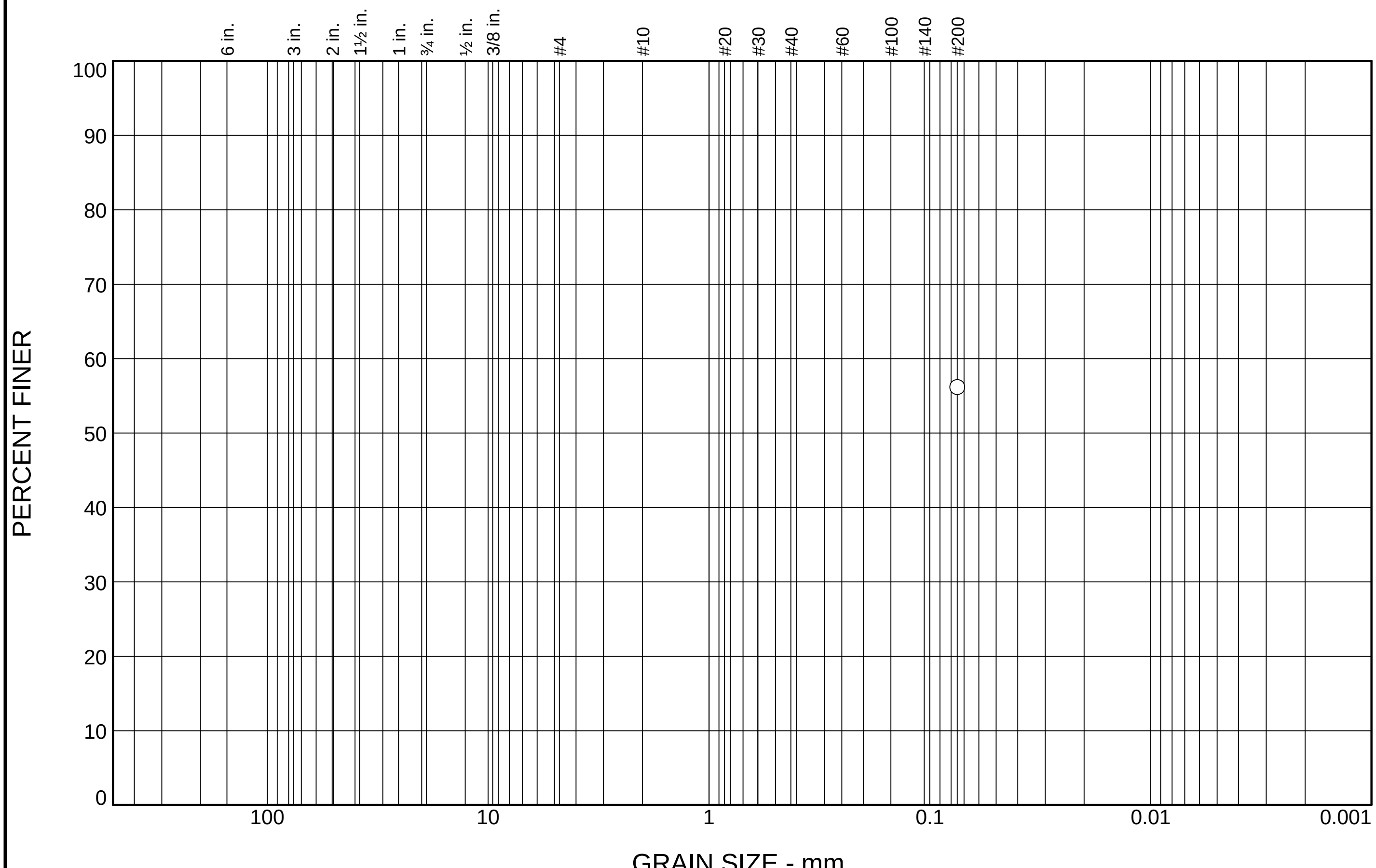
Blackburn Consulting

W. Sacramento, CA

Client: Stantec - Rocklin
Project: LWWTRF Expansion Phase 1&2

Project No: 3228.X **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						56	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	56		

* (no specification provided)

Material Description

SANDY lean CLAY, reddish brown

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Source of Sample: TP4 **Depth:** 0.0-8.5'
Sample Number: Bulk A

Date: 10/31/17

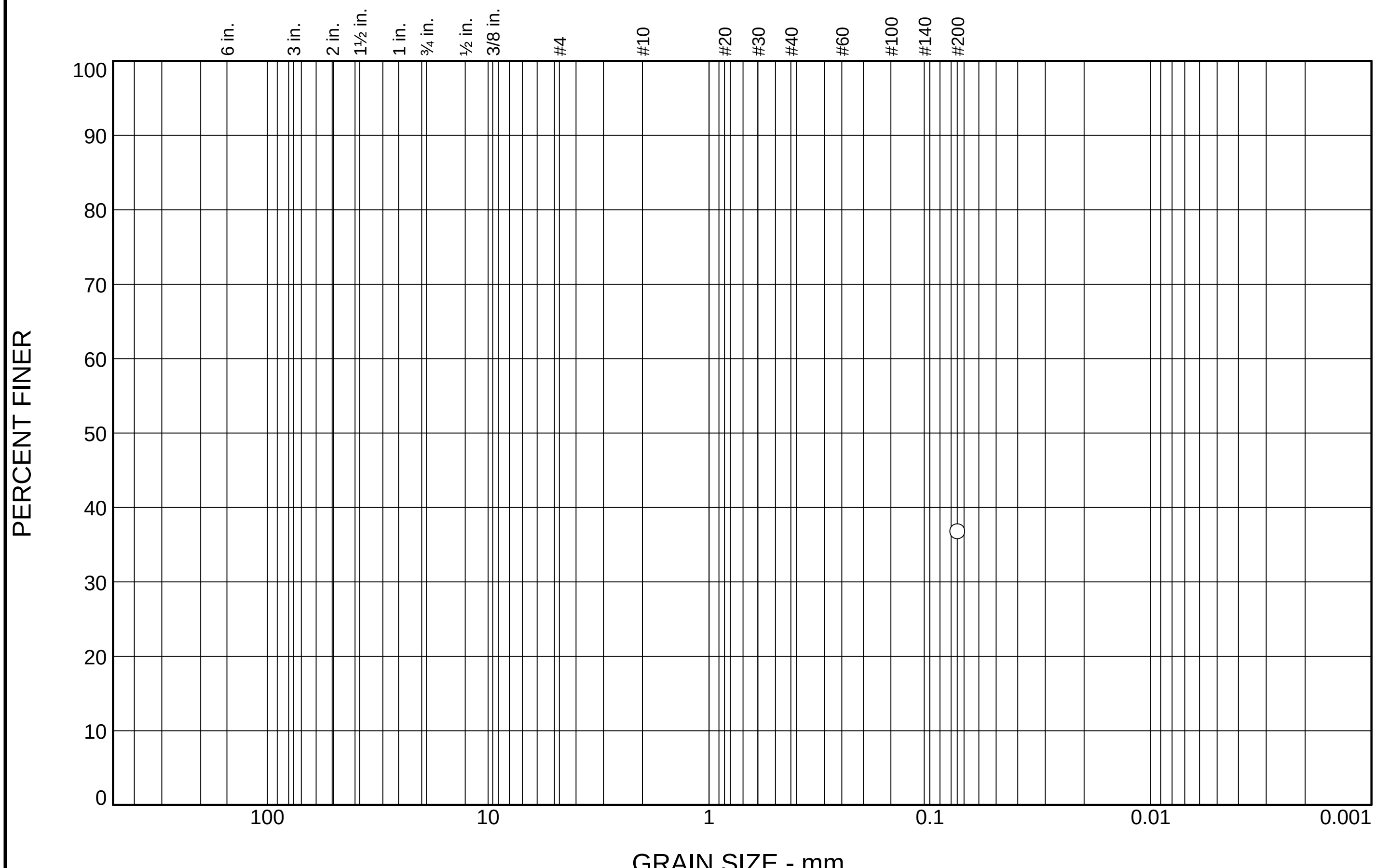
Blackburn Consulting

W. Sacramento, CA

Client: Stantec - Rocklin
Project: LWWTRF Expansion Phase 1&2

Project No: 3228.X **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						37	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	37		

* (no specification provided)

Material Description

CLAYEY SAND, brown

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Source of Sample: B6 **Depth:** 11.0-11.5'
Sample Number: 3C

Date: 11/20/17

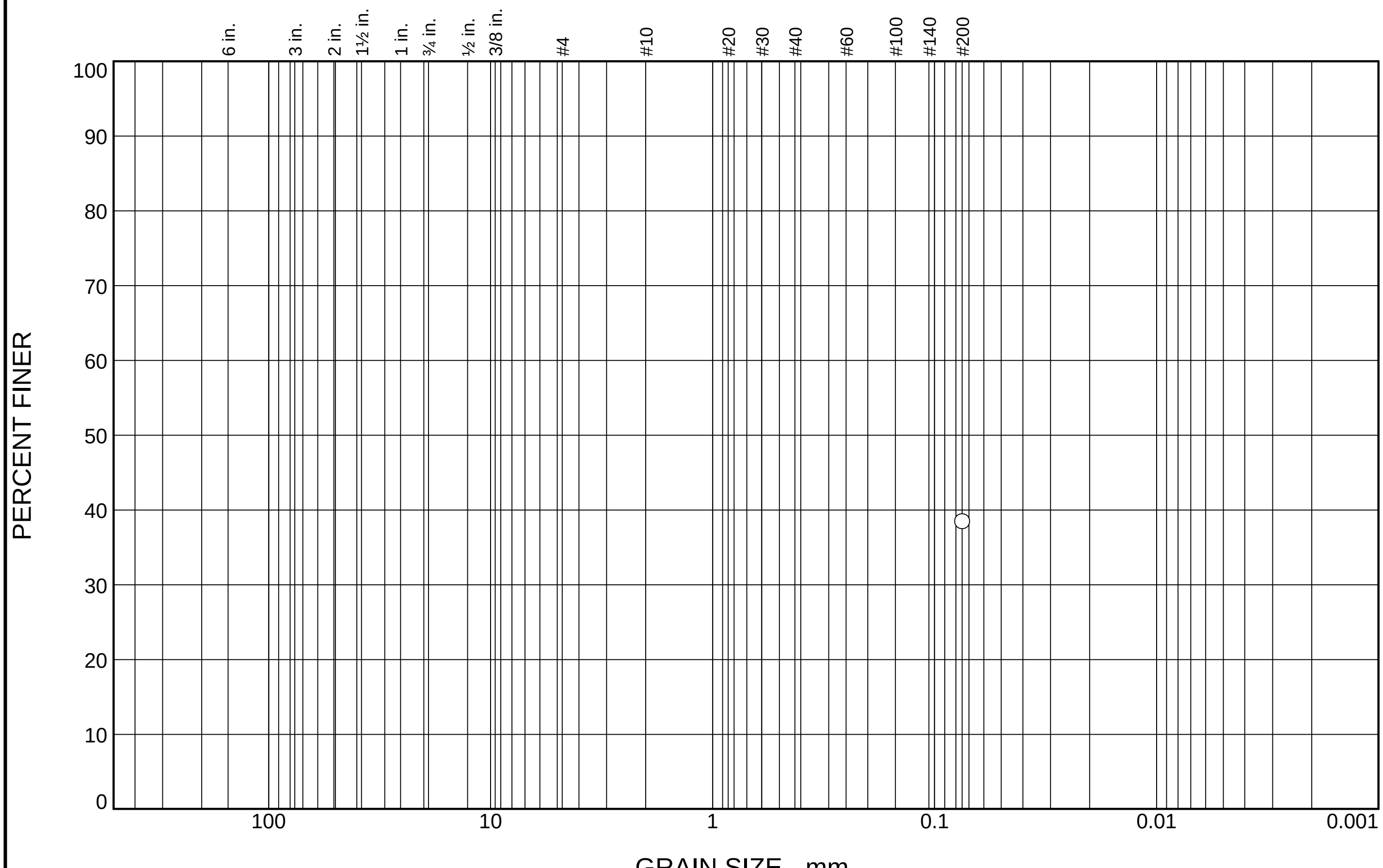
Blackburn Consulting

W. Sacramento, CA

Client: Stantec - Rocklin
Project: LWWTRF Expansion Phase 1&2

Project No: 3228.X **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						38	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	38		

* (no specification provided)

Material Description

CLAYEY SAND, reddish brown

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

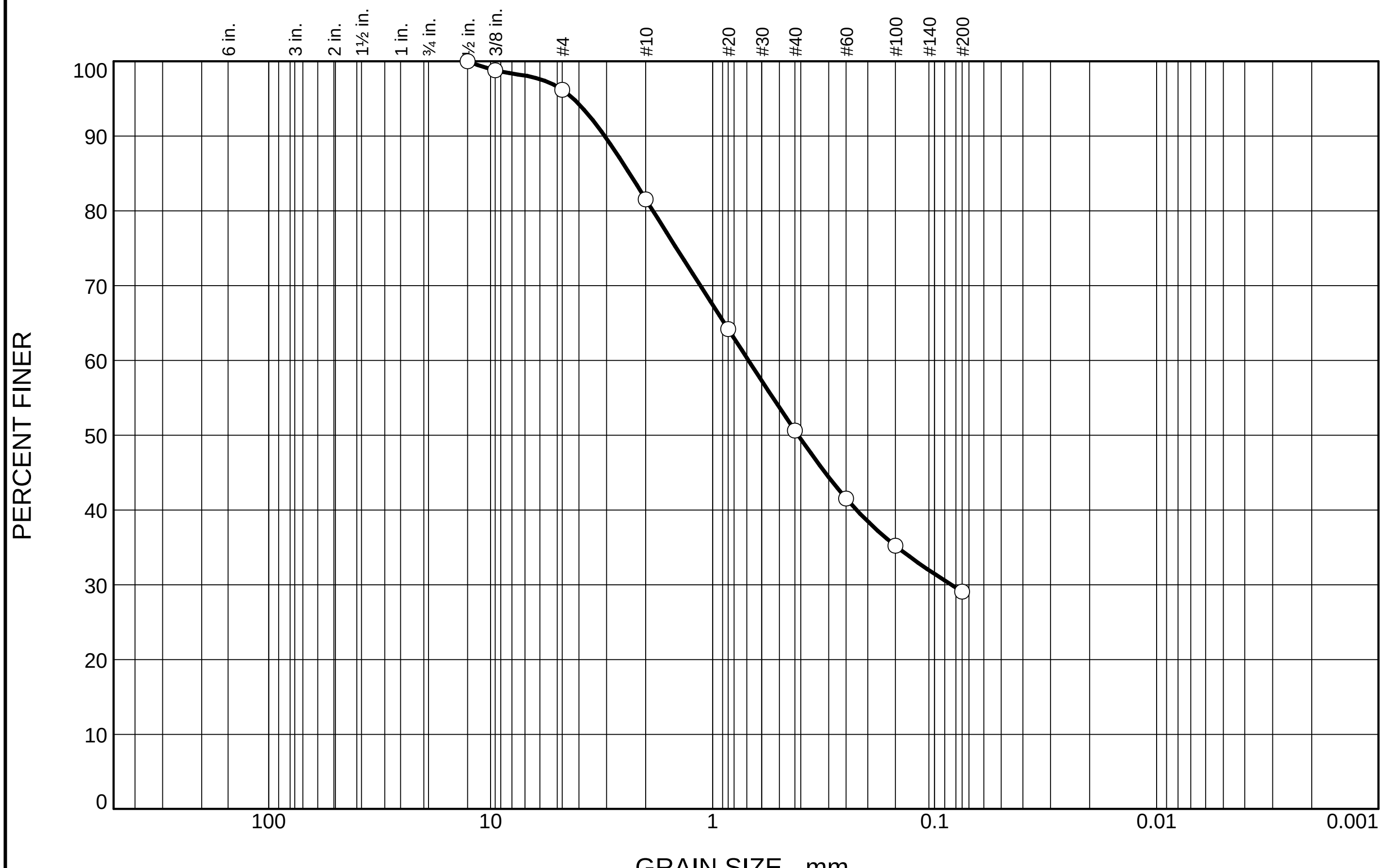
Remarks

Source of Sample: TP7 **Depth:** 0.0-3.0'
Sample Number: Bulk A

Date: 10/31/17

Blackburn Consulting **W. Sacramento, CA**	**Client:** Stantec - Rocklin **Project:** LWWTRF Expansion Phase 1&2 **Project No:** 3228.X **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	4	14	31	22	29	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100		
3/8"	99		
#4	96		
#10	82		
#20	64		
#40	51		
#60	42		
#100	35		
#200	29		

* (no specification provided)

Material Description

CLAYEY SAND, reddish brown

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 3.0623 D_{85}= 2.3670 D_{60}= 0.6894
D_{50}= 0.4113 D_{30}= 0.0840 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Source of Sample: TP8 **Depth:** 3.0-6.0'
Sample Number: Bulk B

Date: 10/31/17

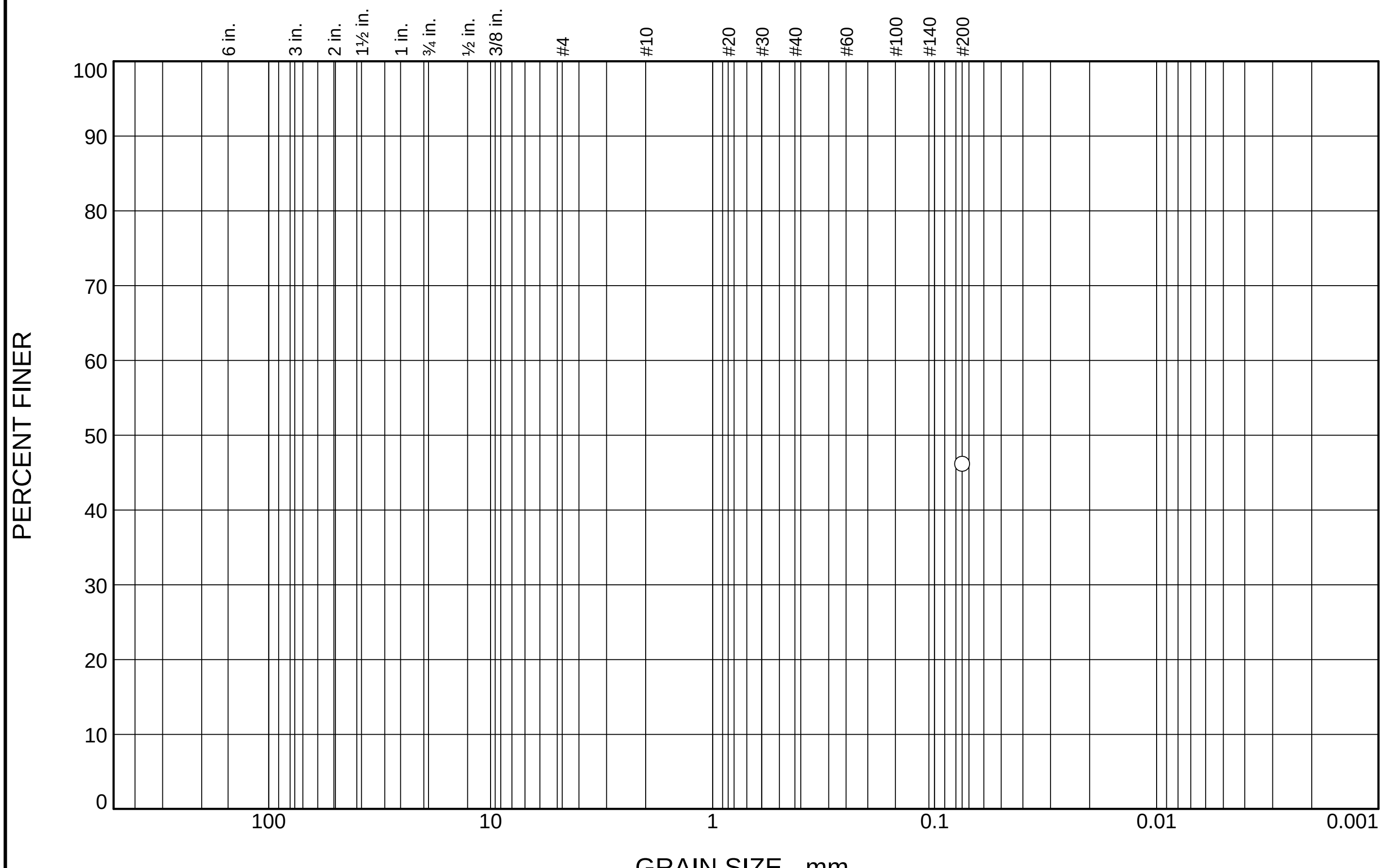
Blackburn Consulting

W. Sacramento, CA

Client: Stantec - Rocklin
Project: LWWTRF Expansion Phase 1&2

Project No: 3228.X **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						46	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	46		

* (no specification provided)

Material Description

CLAYEY SAND, reddish brown

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Source of Sample: TP8 **Depth:** 0.0-3.0'
Sample Number: Bulk A

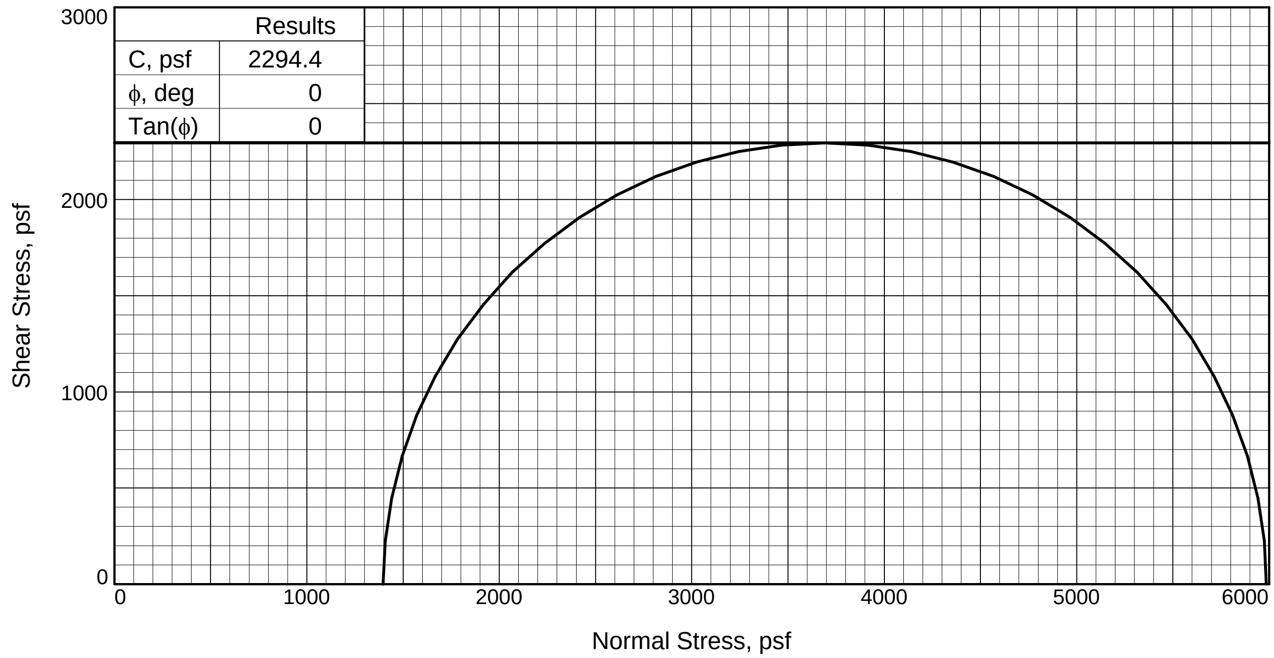
Date: 10/31/17

Blackburn Consulting

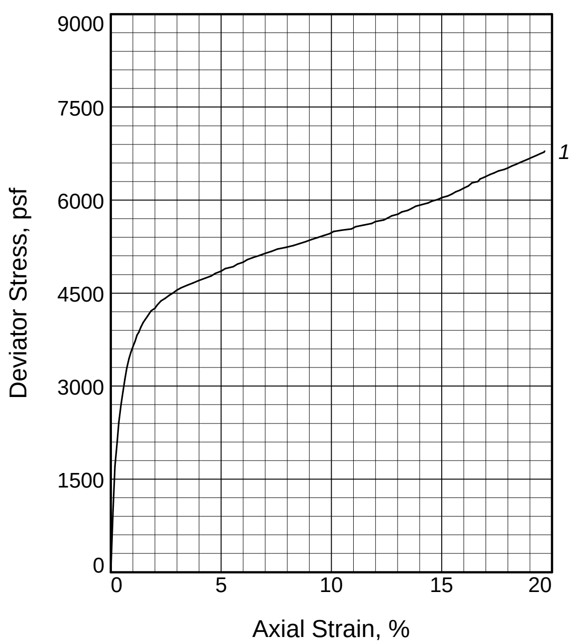
W. Sacramento, CA

Client: Stantec - Rocklin
Project: LWWTRF Expansion Phase 1&2

Project No: 3228.X **Figure**



	Results
C, psf	2294.4
ϕ, deg	0
Tan(ϕ)	0



	Sample No.	1
Initial	Water Content, %	18.7
	Dry Density, pcf	104.7
	Saturation, %	83.0
	Void Ratio	0.6095
	Diameter, in.	2.400
	Height, in.	5.590
At Test	Water Content, %	22.0
	Dry Density, pcf	104.7
	Saturation, %	97.3
	Void Ratio	0.6095
	Diameter, in.	2.400
	Height, in.	5.590
	Strain rate, in./min.	0.056
	Back Pressure, psf	0.0
	Cell Pressure, psf	1396.8
	Fail. Stress, psf	4588.7
	Strain, %	3.2
	Ult. Stress, psf	
	Strain, %	
	$\pm_1$ Failure, psf	5985.5
	$\pm_3$ Failure, psf	1396.8

Type of Test:
Unconsolidated Undrained
Sample Type: 2.4" Mod Cal
Description: Lean CLAY, brown

Assumed Specific Gravity= 2.70
Remarks:

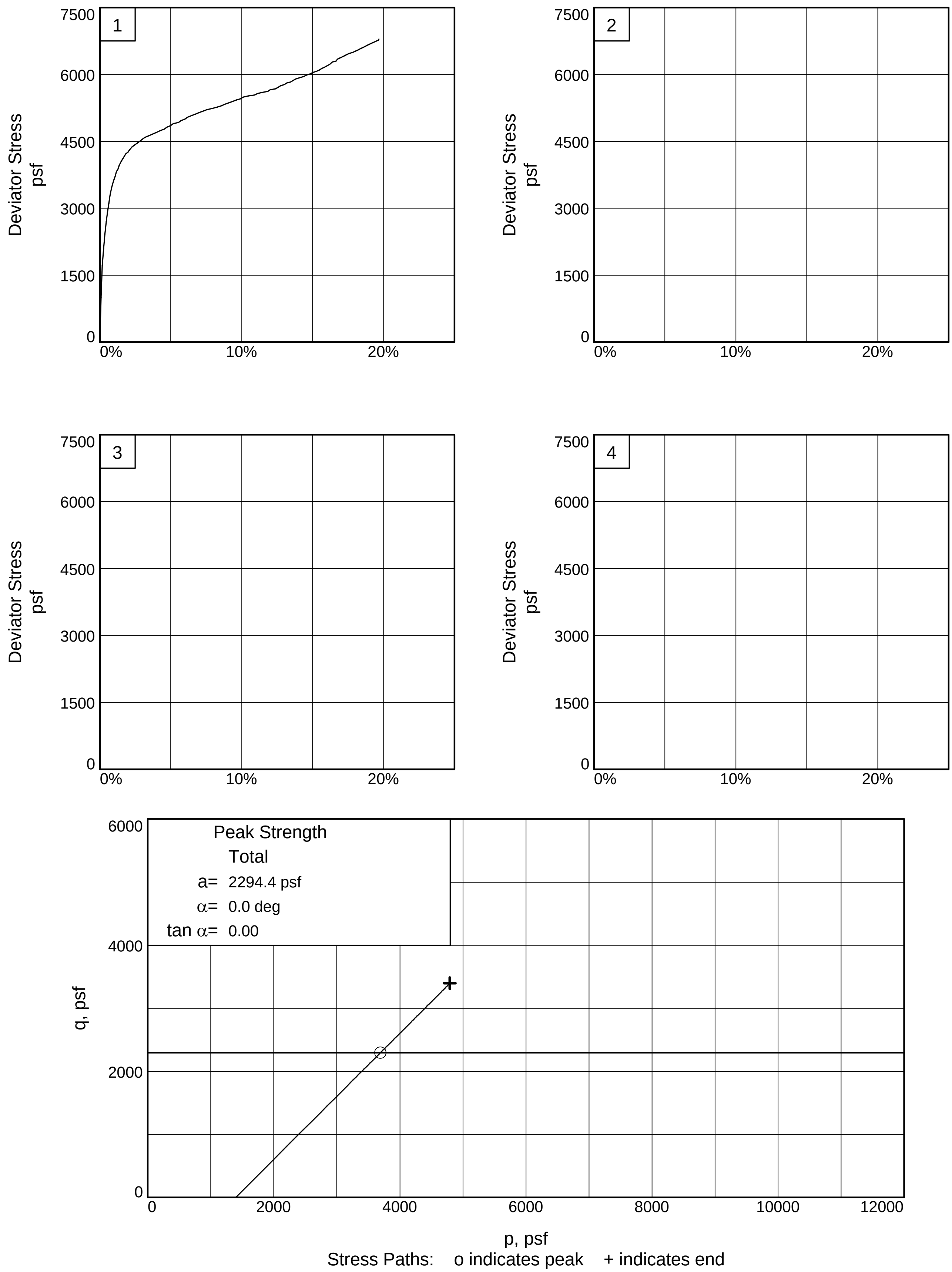
Client: Stantec - Rocklin

Project: LWWTRF Expansion Phase 1&2

Source of Sample: B1 **Depth:** 5.75-6.25
Sample Number: 2C
Proj. No.: 3228.X **Date Sampled:** 10/6/17

TRIAXIAL SHEAR TEST REPORT
Blackburn Consulting
W. Sacramento, CA

Figure ________



Client: Stantec - Rocklin

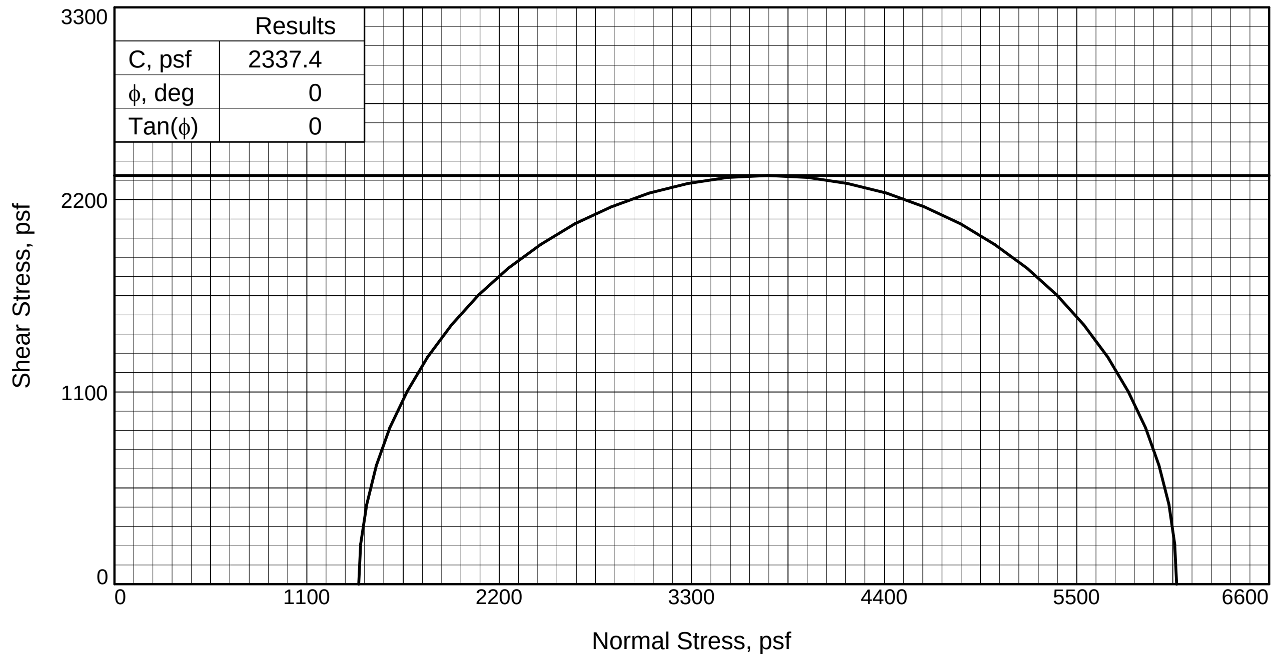
Project: LWWTRF Expansion Phase 1&2

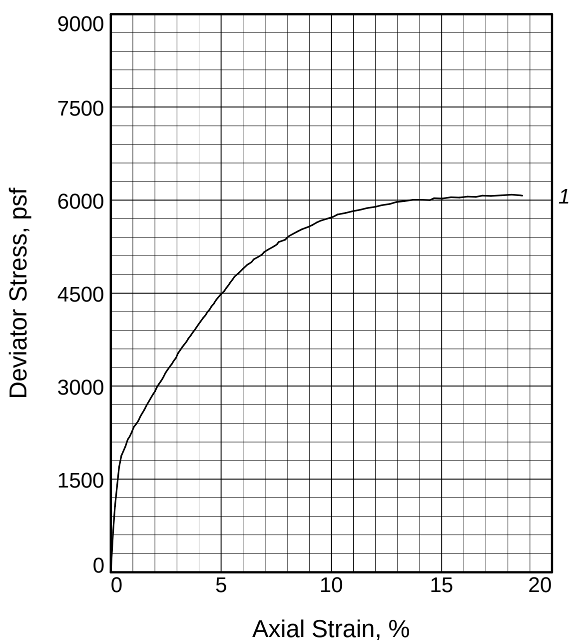
Source of Sample: B1 **Depth:** 5.75-6.25 **Sample Number:** 2C

Project No.: 3228.X **Figure** ________

Blackburn Consulting

Results	
C, psf	2337.4
ϕ, deg	0
Tan(ϕ)	0





Sample No.		1
Initial	Water Content, %	24.0
	Dry Density, pcf	102.2
	Saturation, %	99.9
	Void Ratio	0.6491
	Diameter, in.	2.390
	Height, in.	5.343
At Test	Water Content, %	25.3
	Dry Density, pcf	102.2
	Saturation, %	105.1
	Void Ratio	0.6491
	Diameter, in.	2.390
	Height, in.	5.343
Strain rate, in./min.		0.053
Back Pressure, psf		0.0
Cell Pressure, psf		1396.8
Fail. Stress, psf		4674.9
Strain, %		5.4
Ult. Stress, psf		
Strain, %		
$\pm_1$ Failure, psf		6071.7
$\pm_3$ Failure, psf		1396.8

Type of Test:
Unconsolidated Undrained

Sample Type: 2.4" Mod Cal

Description: Lean CLAY, reddish brown

Assumed Specific Gravity= 2.70

Remarks:

Figure ________

Client: Stantec - Rocklin

Project: LWWTRF Expansion Phase 1&2

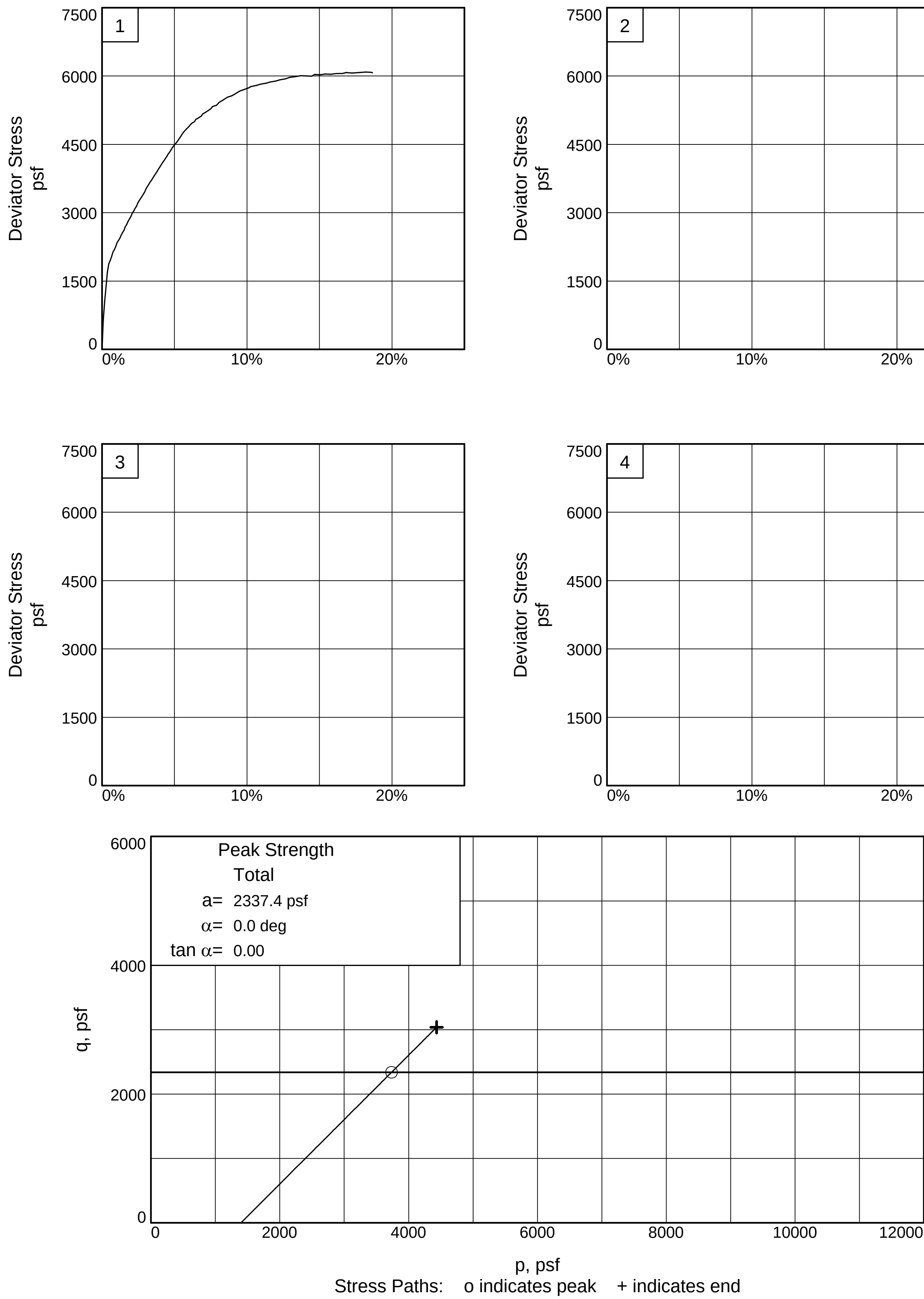
Source of Sample: B3 **Depth:** 21.0-21.5'

Sample Number: 5C

Proj. No.: 3228.X **Date Sampled:** 10/6/17

TRIAXIAL SHEAR TEST REPORT

Blackburn Consulting
W. Sacramento, CA

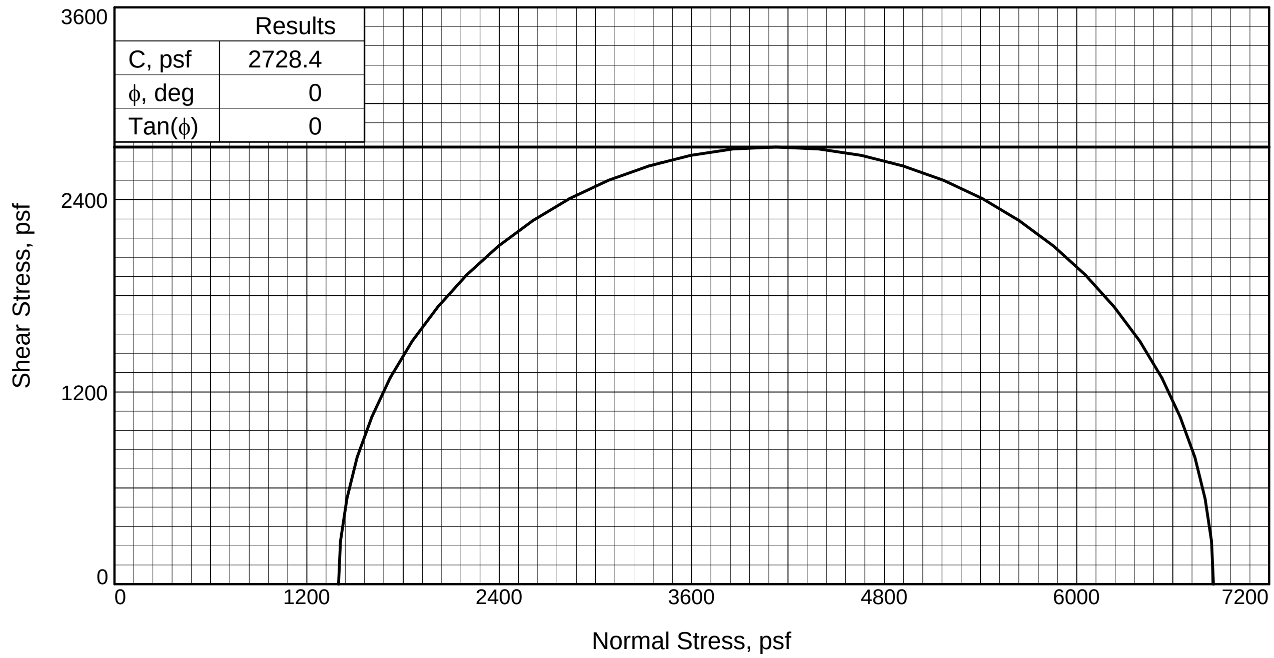


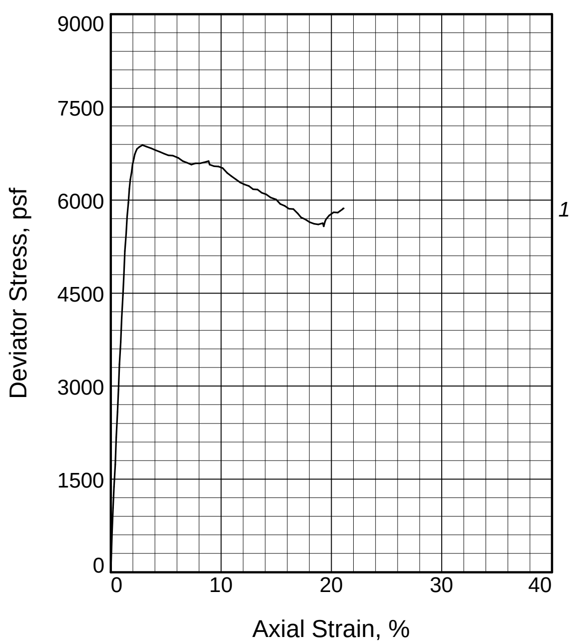
Client: Stantec - Rocklin

Project: LWWTRF Expansion Phase 1&2

Source of Sample: B3 **Depth:** 21.0-21.5' **Sample Number:** 5C

Project No.: 3228.X **Figure** ________ **Blackburn Consulting**





Sample No.	1
Initial	
Water Content, %	18.8
Dry Density, pcf	97.5
Saturation, %	69.8
Void Ratio	0.7287
Diameter, in.	2.417
Height, in.	5.162
At Test	
Water Content, %	17.3
Dry Density, pcf	97.5
Saturation, %	63.9
Void Ratio	0.7287
Diameter, in.	2.417
Height, in.	5.162
Strain rate, in./min.	0.052
Back Pressure, psf	0.0
Cell Pressure, psf	1396.8
Fail. Stress, psf	5456.8
Strain, %	1.4
Ult. Stress, psf	
Strain, %	
±1 Failure, psf	6853.6
±3 Failure, psf	1396.8

Type of Test:
Unconsolidated Undrained

Sample Type: 2.4" Mod Cal

Description: SANDY lean CLAY, yellowish brown

Assumed Specific Gravity= 2.70

Remarks:

Figure ________

Client: Stantec - Rocklin

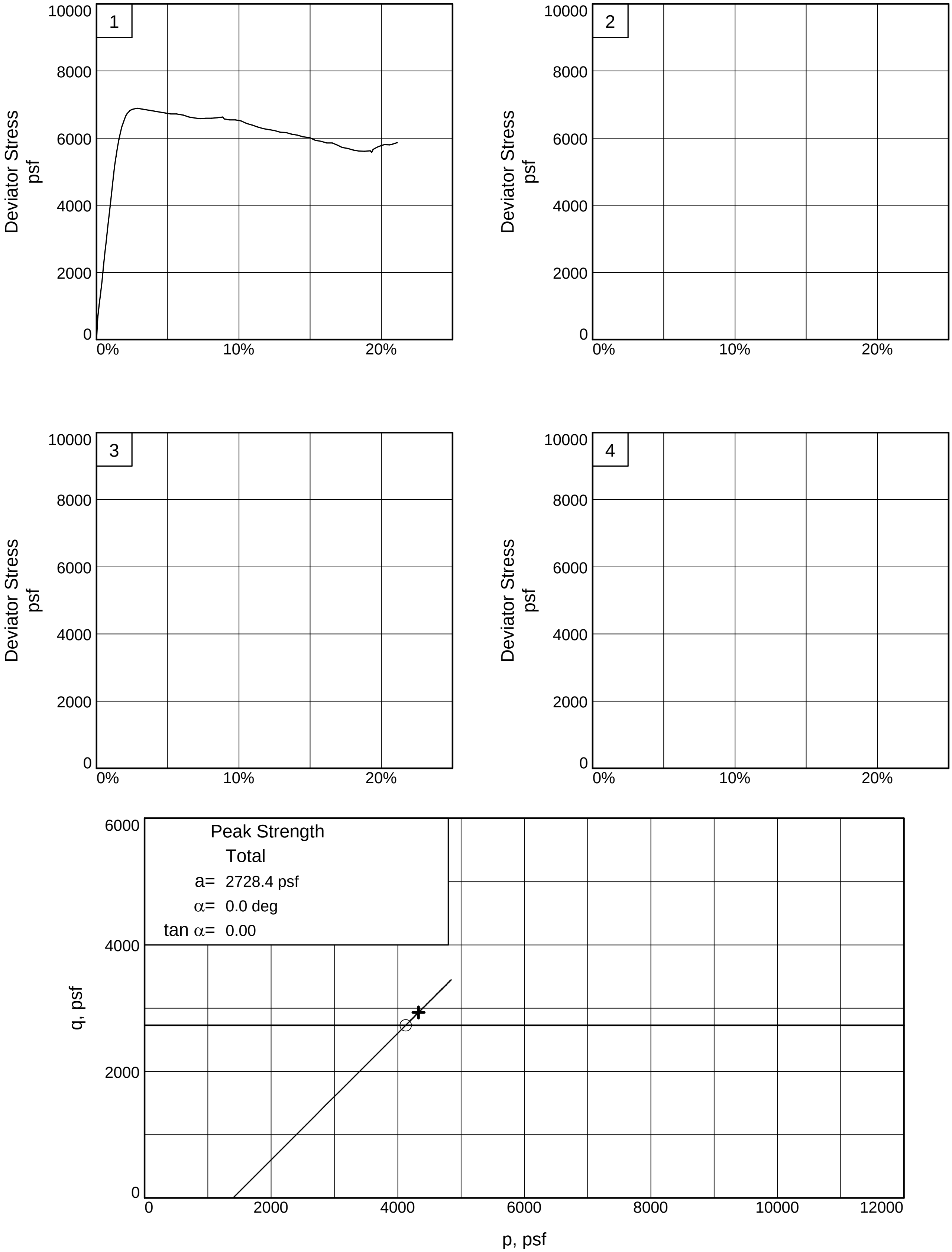
Project: LWWTRF Expansion Phase 1&2

Source of Sample: B5 **Depth:** 6.0-6.5'

Sample Number: 2C

Proj. No.: 3228.X **Date Sampled:** 10/6/17

TRIAXIAL SHEAR TEST REPORT
Blackburn Consulting
W. Sacramento, CA



Stress Paths: o indicates peak + indicates end

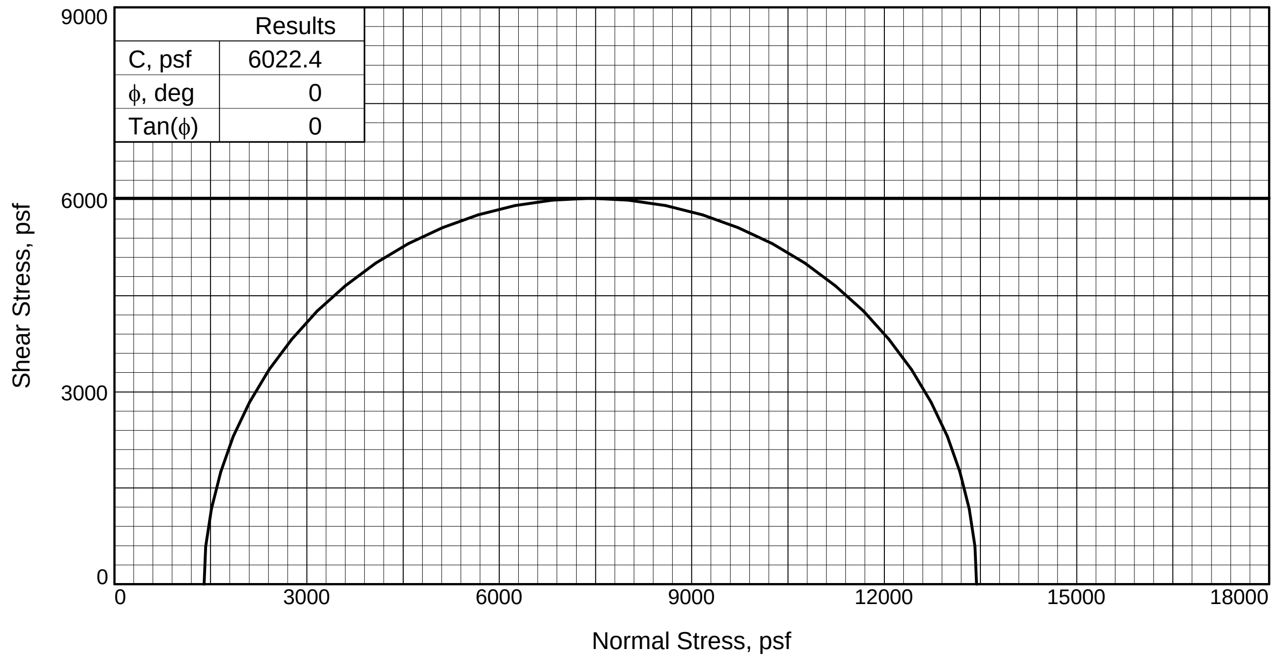
Client: Stantec - Rocklin

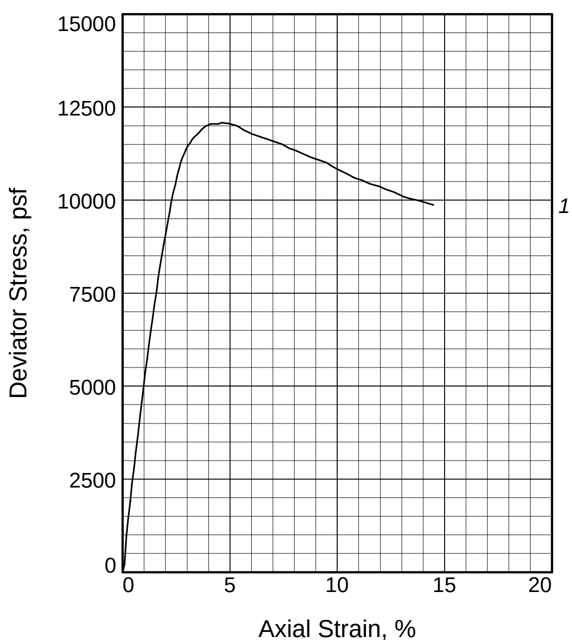
Project: LWWTRF Expansion Phase 1&2

Source of Sample: B5 **Depth:** 6.0-6.5' **Sample Number:** 2C

Project No.: 3228.X **Figure** ________

Blackburn Consulting





Sample No.		1
Initial	Water Content, %	34.0
	Dry Density, pcf	88.1
	Saturation, %	100.4
	Void Ratio	0.9137
	Diameter, in.	2.408
	Height, in.	5.833
At Test	Water Content, %	33.7
	Dry Density, pcf	88.1
	Saturation, %	99.5
	Void Ratio	0.9137
	Diameter, in.	2.408
	Height, in.	5.833
Strain rate, in./min.		0.058
Back Pressure, psf		0.0
Cell Pressure, psf		1396.8
Fail. Stress, psf		12044.8
Strain, %		4.1
Ult. Stress, psf		
Strain, %		
$\pm_1$ Failure, psf		13441.6
$\pm_3$ Failure, psf		1396.8

Type of Test:
Unconsolidated Undrained

Sample Type: 2.4" Mod Cal

Description: Lean CLAY, light yellowish brown

Assumed Specific Gravity= 2.70

Remarks:

Figure ________

Client: Stantec - Rocklin

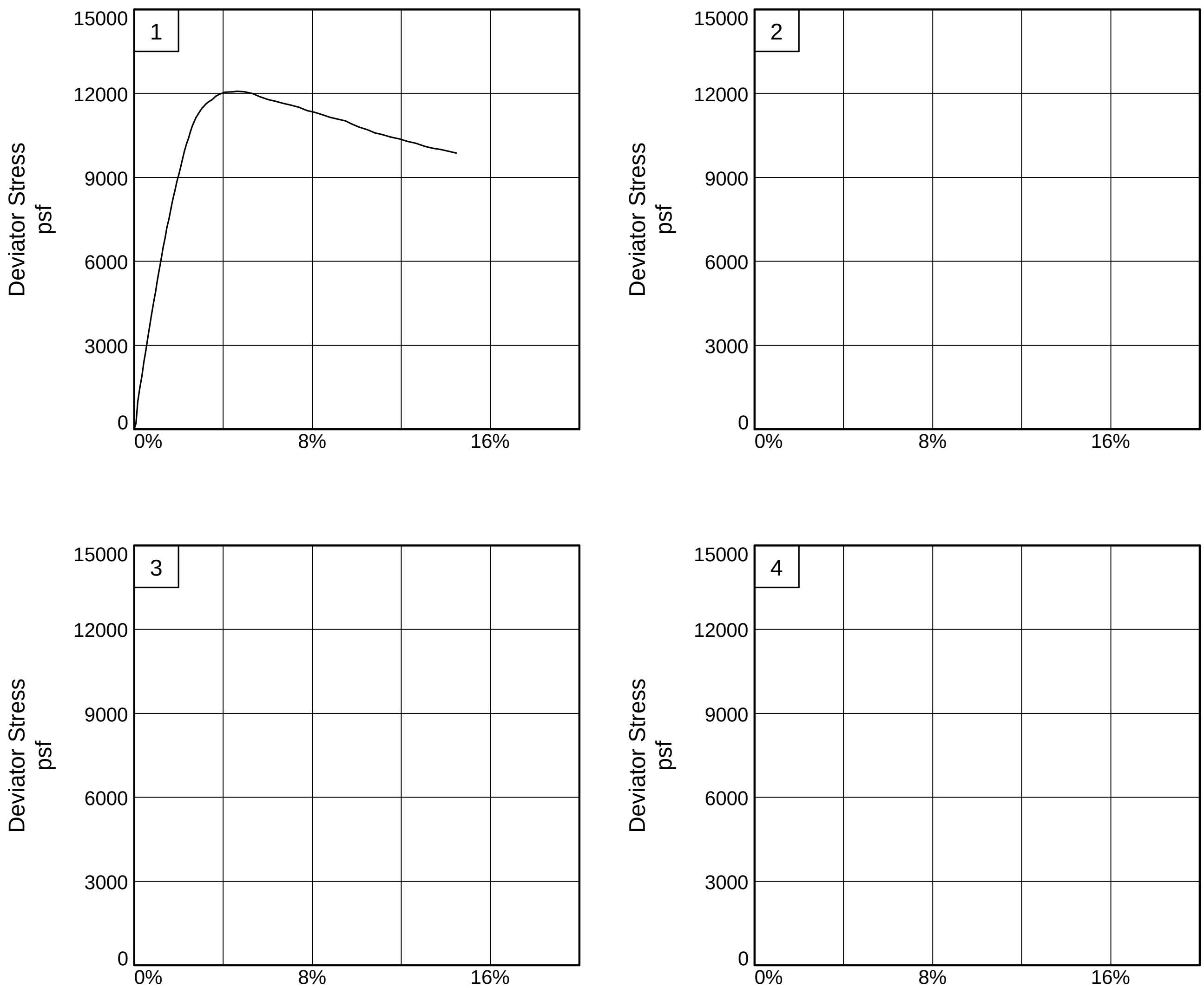
Project: LWWTRF Expansion Phase 1&2

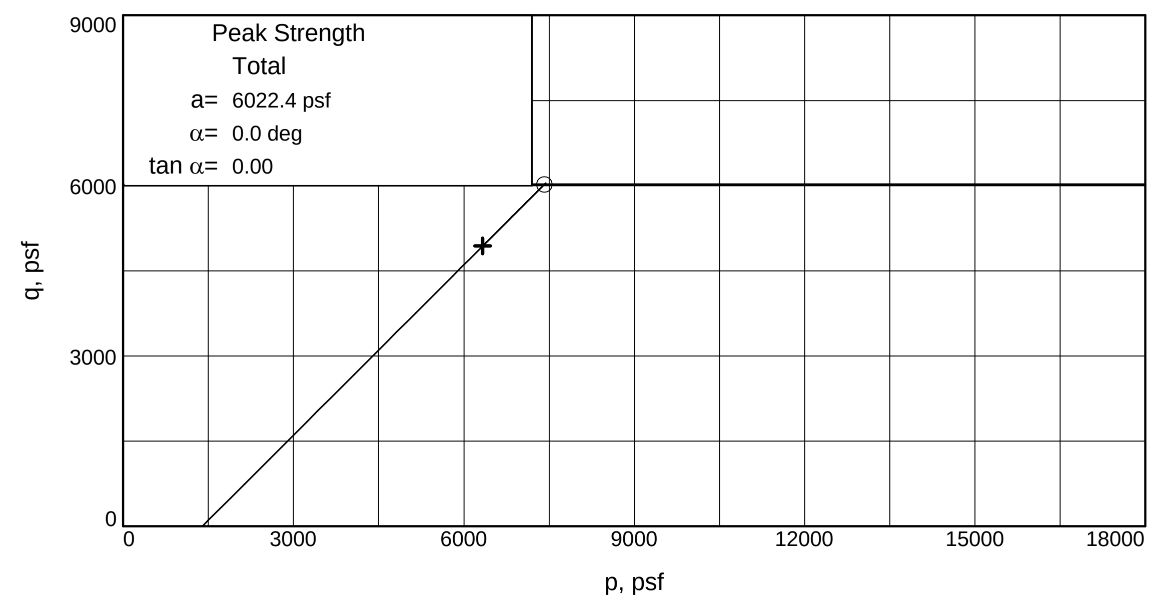
Source of Sample: B5 **Depth:** 11.0-11.5'

Sample Number: 3C

Proj. No.: 3228.X **Date Sampled:** 10/6/17

TRIAXIAL SHEAR TEST REPORT
Blackburn Consulting
W. Sacramento, CA





Stress Paths: o indicates peak + indicates end

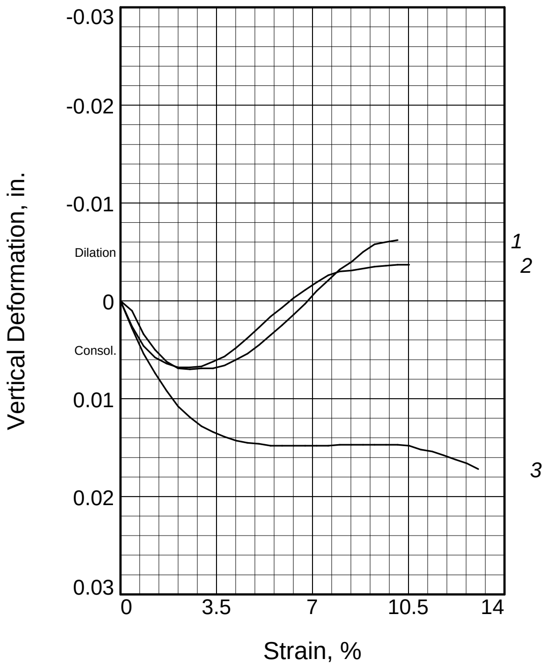
Client: Stantec - Rocklin

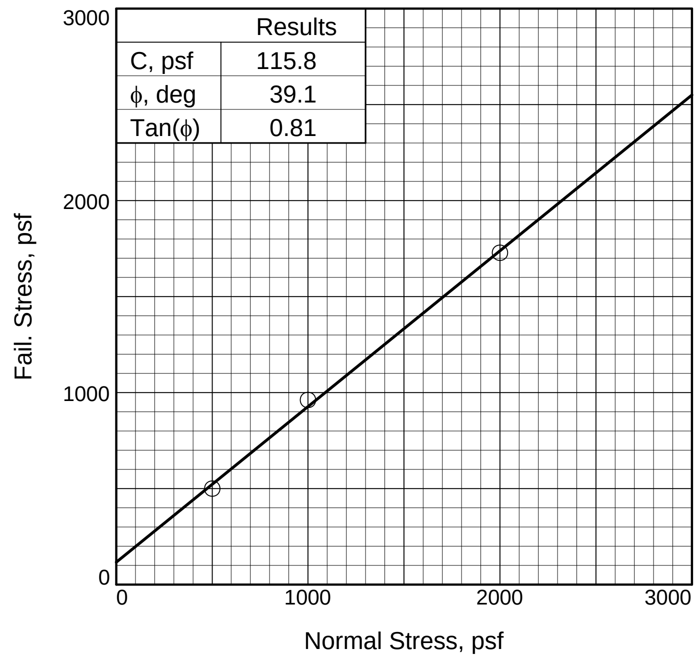
Project: LWWTRF Expansion Phase 1&2

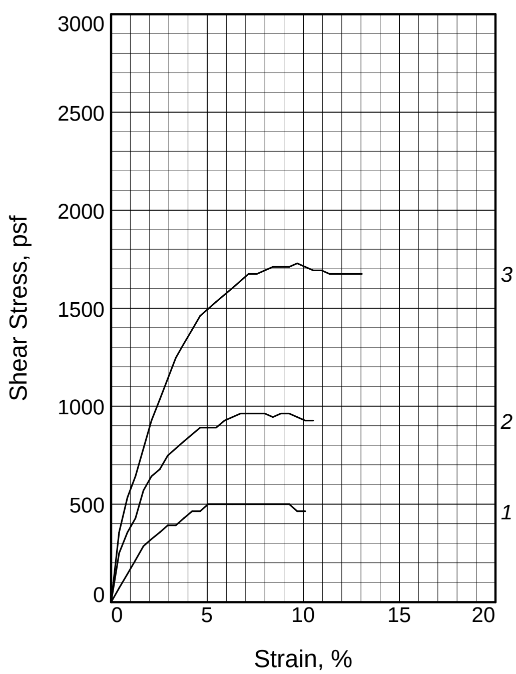
Source of Sample: B5 **Depth:** 11.0-11.5' **Sample Number:** 3C

Project No.: 3228.X **Figure** ________

Blackburn Consulting







	Sample No.	1	2	3
Initial	Water Content, %	5.9	5.9	5.9
	Dry Density, pcf	102.6	112.5	100.0
	Saturation, %	24.7	31.9	23.2
	Void Ratio	0.6431	0.4983	0.6852
	Diameter, in.	2.375	2.375	2.375
	Height, in.	0.950	0.950	0.950
At Test	Water Content, %	19.3	17.0	19.3
	Dry Density, pcf	104.7	115.5	104.9
	Saturation, %	85.5	100.0	86.0
	Void Ratio	0.6097	0.4596	0.6066
	Diameter, in.	2.375	2.375	2.375
	Height, in.	0.931	0.925	0.906
	Normal Stress, psf	500.0	1000.0	2000.0
	Fail. Stress, psf	498.8	961.9	1727.8
	Strain, %	5.1	6.7	9.7
	Ult. Stress, psf			
	Strain, %			
	Strain rate, in./min.	0.006	0.006	0.006

Sample Type: Undisturbed 2.4" Mod Cal

Description: Poorly-graded SAND with CLAY, strong brown

Assumed Specific Gravity= 2.70

Remarks:

Figure ________

Client: Stantec - Rocklin

Project: LWWTRF Expansion Phase 1&2

Source of Sample: B3 **Depth:** 6.0-6.5'

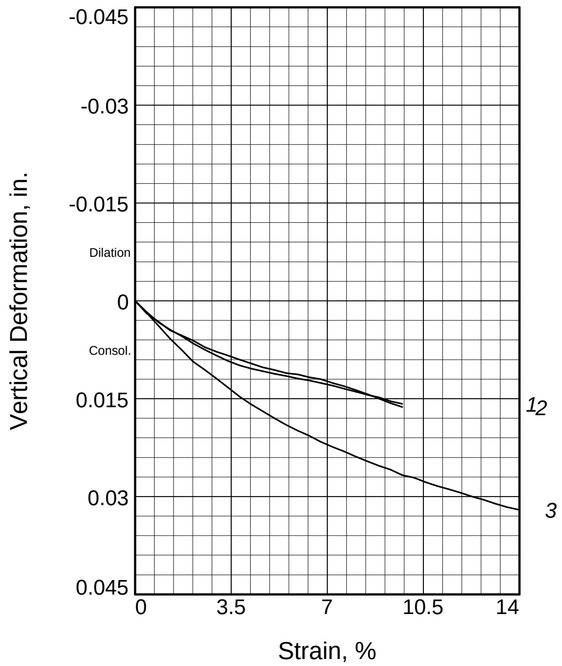
Sample Number: 2C

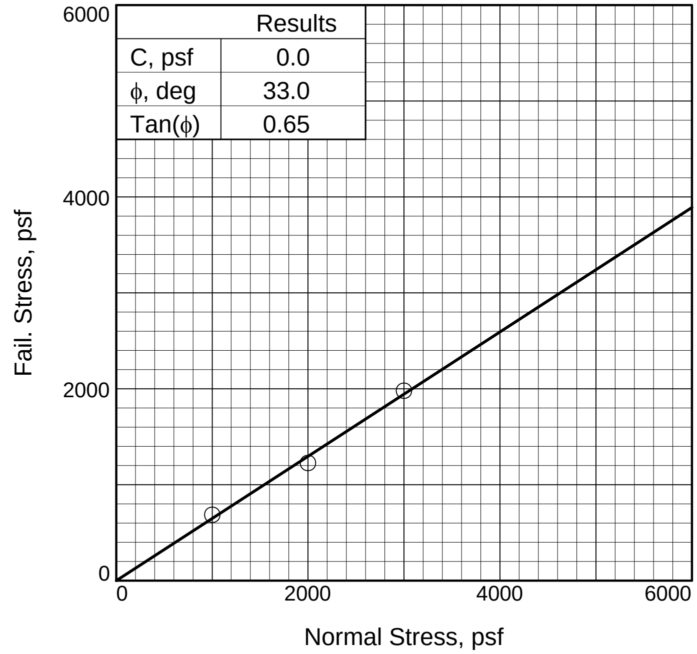
Proj. No.: 3228.X **Date Sampled:** 10/6/2017

DIRECT SHEAR TEST REPORT

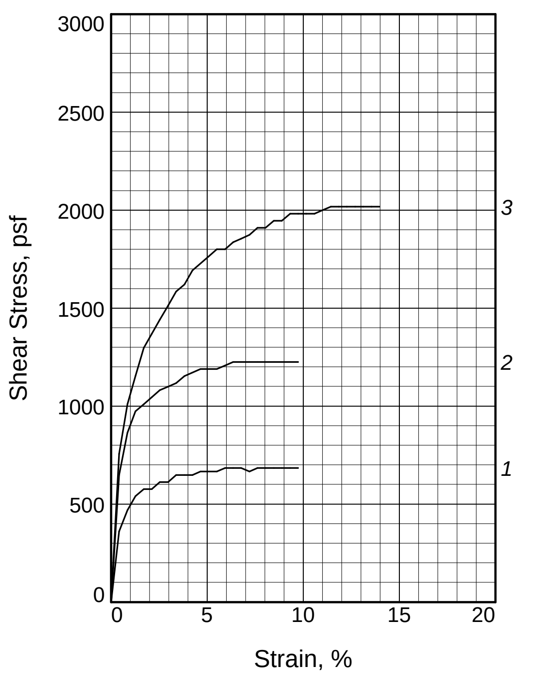
Blackburn Consulting

W. Sacramento, CA





Results	
C, psf	0.0
ϕ, deg	33.0
Tan(ϕ)	0.65



	Sample No.	1	2	3
Initial	Water Content, %	14.2	14.2	14.2
	Dry Density, pcf	111.5	111.5	111.6
	Saturation, %	75.0	75.2	75.4
	Void Ratio	0.5122	0.5111	0.5100
	Diameter, in.	2.362	2.362	2.362
	Height, in.	0.945	0.945	0.945
At Test	Water Content, %	18.2	18.5	18.9
	Dry Density, pcf	112.8	112.2	111.6
	Saturation, %	99.7	99.8	99.9
	Void Ratio	0.4938	0.5018	0.5100
	Diameter, in.	2.362	2.362	2.362
	Height, in.	0.934	0.939	0.945
	Normal Stress, psf	1000.0	2000.0	3000.0
	Fail. Stress, psf	684.3	1224.6	1981.0
	Strain, %	5.9	6.4	9.3
	Ult. Stress, psf			
	Strain, %			
	Strain rate, in./min.	0.007	0.007	0.007

Sample Type: Remold

Description: SANDY lean CLAY, reddish brown

Assumed Specific Gravity= 2.70

Remarks:

Figure ________

Client: Stantec - Rocklin

Project: LWWTRF Expansion Phase 1&2

Source of Sample: TP2 **Depth:** 0.0-8.5'

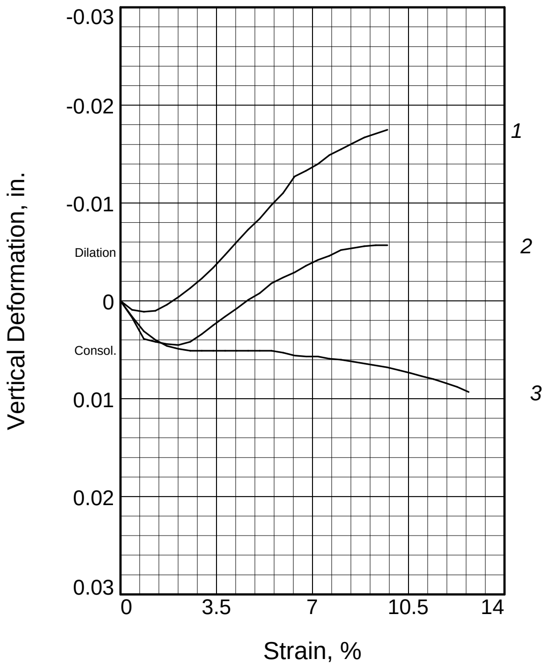
Sample Number: Bulk A

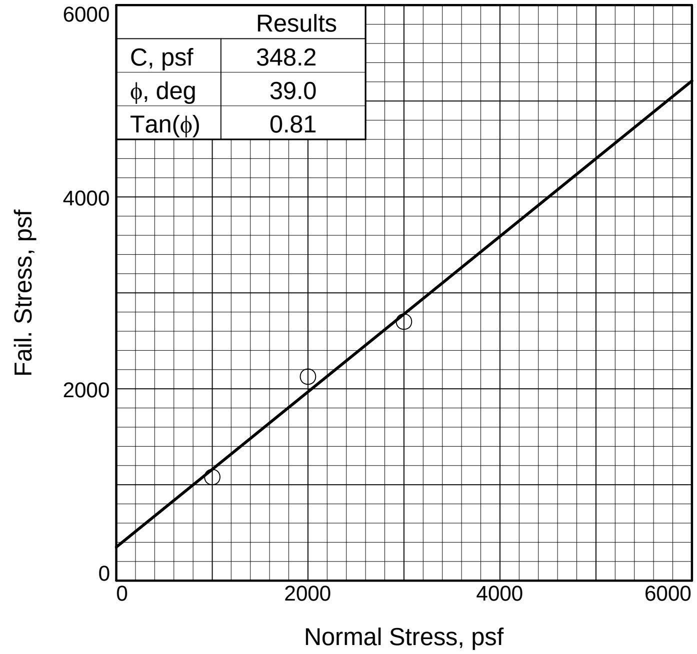
Proj. No.: 3228.X **Date Sampled:** 10/31/17

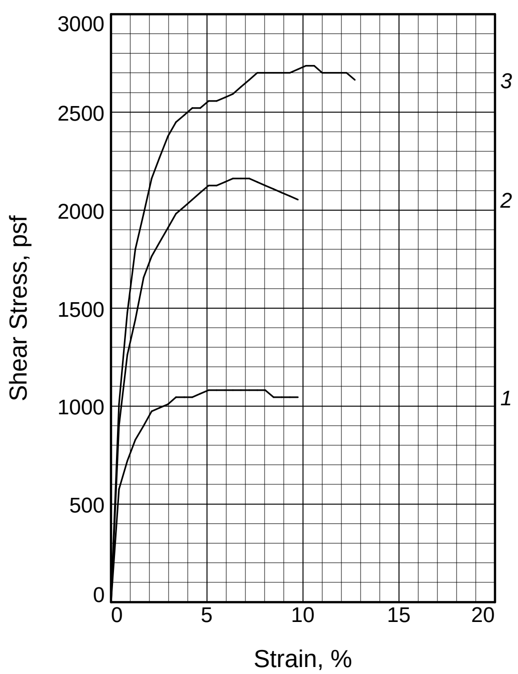
DIRECT SHEAR TEST REPORT

Blackburn Consulting

W. Sacramento, CA







	Sample No.	1	2	3
Initial	Water Content, %	13.7	13.7	13.7
	Dry Density, pcf	114.3	114.1	114.3
	Saturation, %	78.1	77.5	78.1
	Void Ratio	0.4744	0.4776	0.4744
	Diameter, in.	2.362	2.362	2.362
	Height, in.	0.945	0.945	0.945
At Test	Water Content, %	17.4	17.3	16.8
	Dry Density, pcf	114.7	114.8	115.9
	Saturation, %	100.0	99.7	100.0
	Void Ratio	0.4691	0.4679	0.4542
	Diameter, in.	2.362	2.362	2.362
	Height, in.	0.942	0.939	0.932
	Normal Stress, psf	1000.0	2000.0	3000.0
	Fail. Stress, psf	1080.5	2125.1	2701.4
	Strain, %	5.1	5.1	7.6
	Ult. Stress, psf			
	Strain, %			
	Strain rate, in./min.	0.007	0.007	0.007

Sample Type: Remold

Description: SANDY lean CLAY, reddish brown

Assumed Specific Gravity= 2.70

Remarks:

Figure ________

Client: Stantec - Rocklin

Project: LWWTRF Expansion Phase 1&2

Source of Sample: TP4 **Depth:** 0.0-8.5'

Sample Number: Bulk A

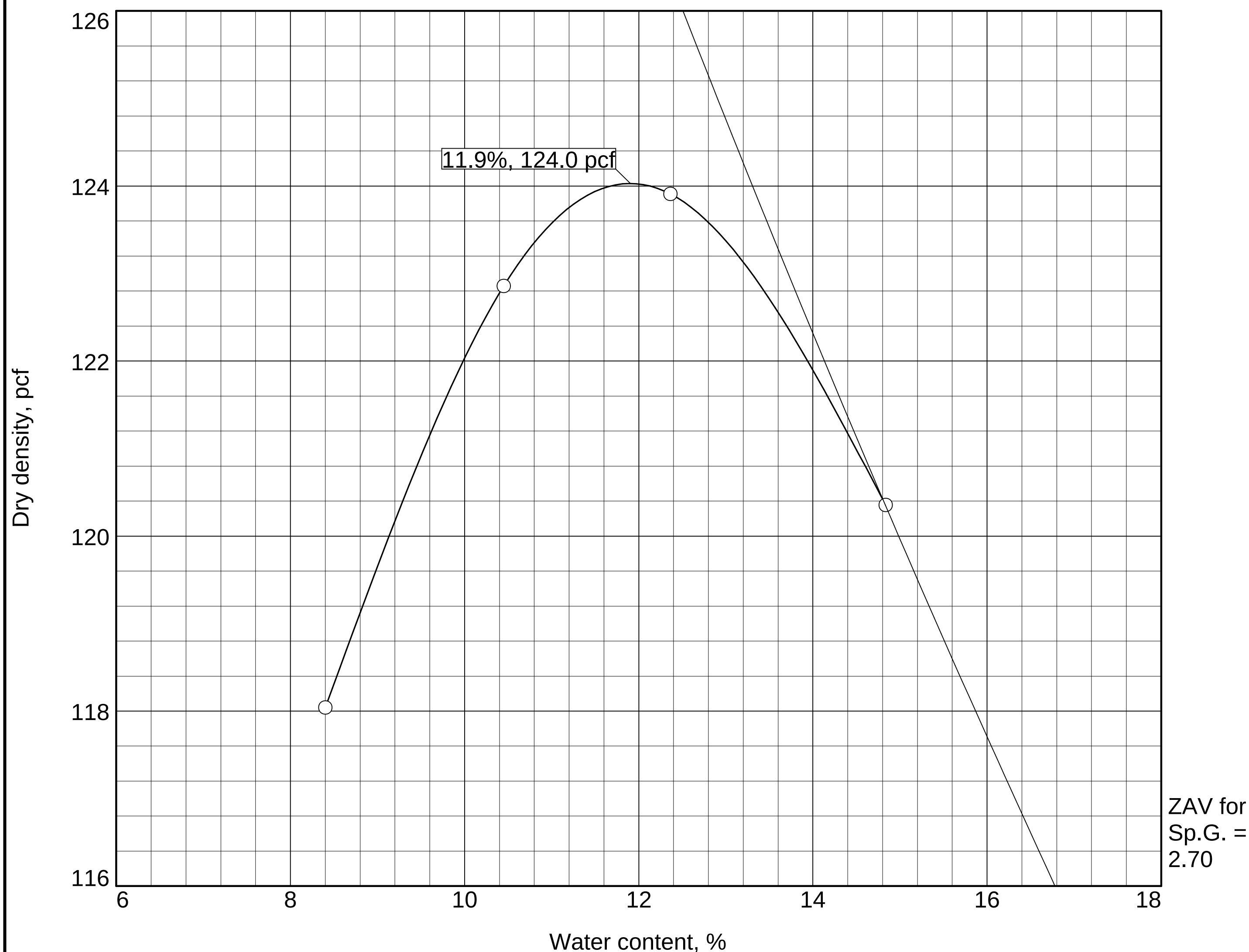
Proj. No.: 3228.X **Date Sampled:** 10/31/17

DIRECT SHEAR TEST REPORT

Blackburn Consulting

W. Sacramento, CA

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified, manual rammer, dry prep method

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
0.0-8.5'				2.70				

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 124.0 pcf Optimum moisture = 11.9 %	SANDY lean CLAY, reddish brown

Project No. 3228.X **Client:** Stantec - Rocklin

Project: LWWTRF Expansion Phase 1&2

○ **Source of Sample:** TP2 **Sample Number:** Bulk A

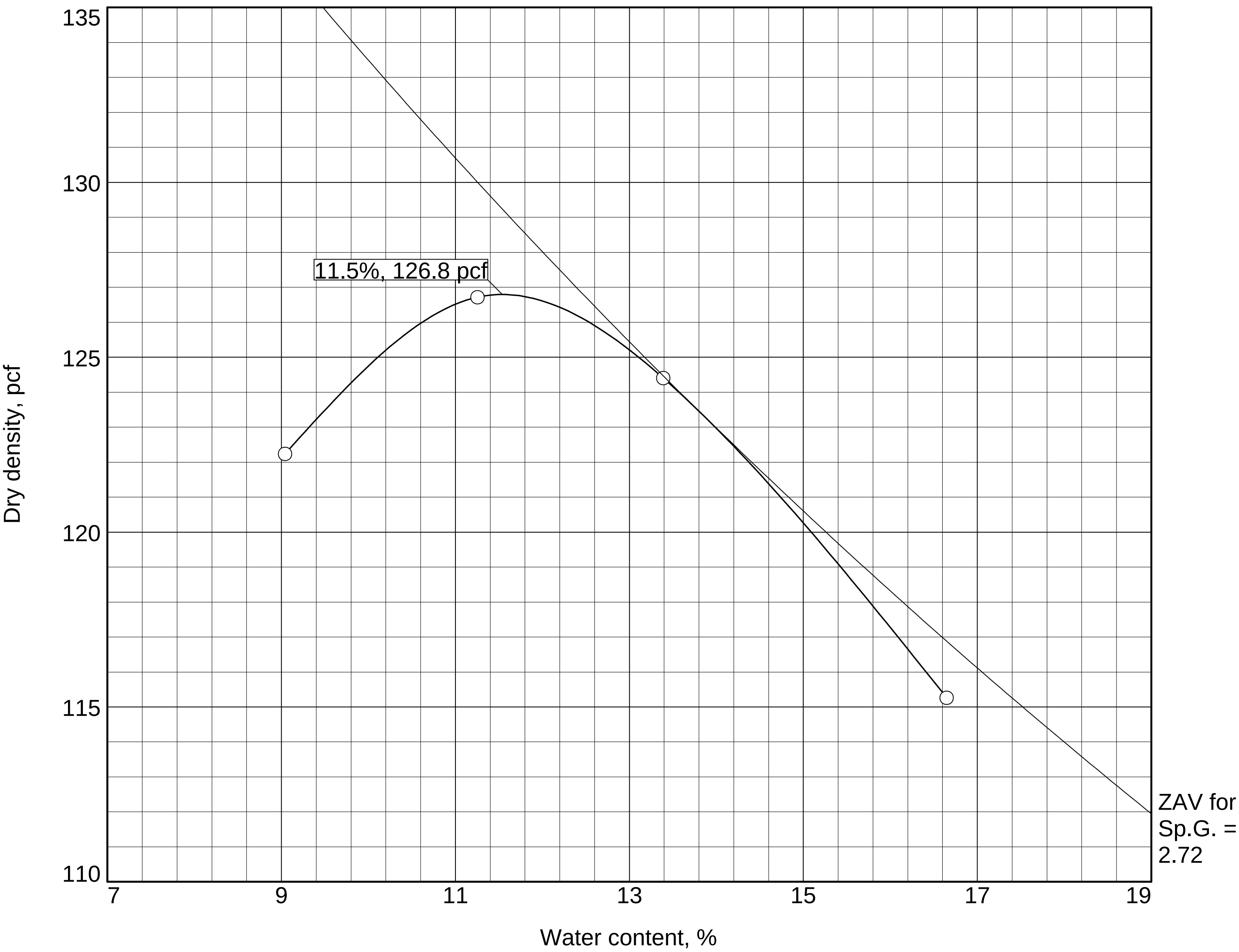
Remarks:

Blackburn Consulting

W. Sacramento, CA

Figure

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified, manual rammer, dry prep method

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
0.0-8.5'				2.72			1.0	56

TEST RESULTS

Maximum dry density = 126.8 pcf

Optimum moisture = 11.5 %

MATERIAL DESCRIPTION

SANDY lean CLAY, reddish brown

Project No. 3228.X **Client:** Stantec - Rocklin

Project: LWWTRF Expansion Phase 1&2

○ **Source of Sample:** TP4 **Sample Number:** Bulk A

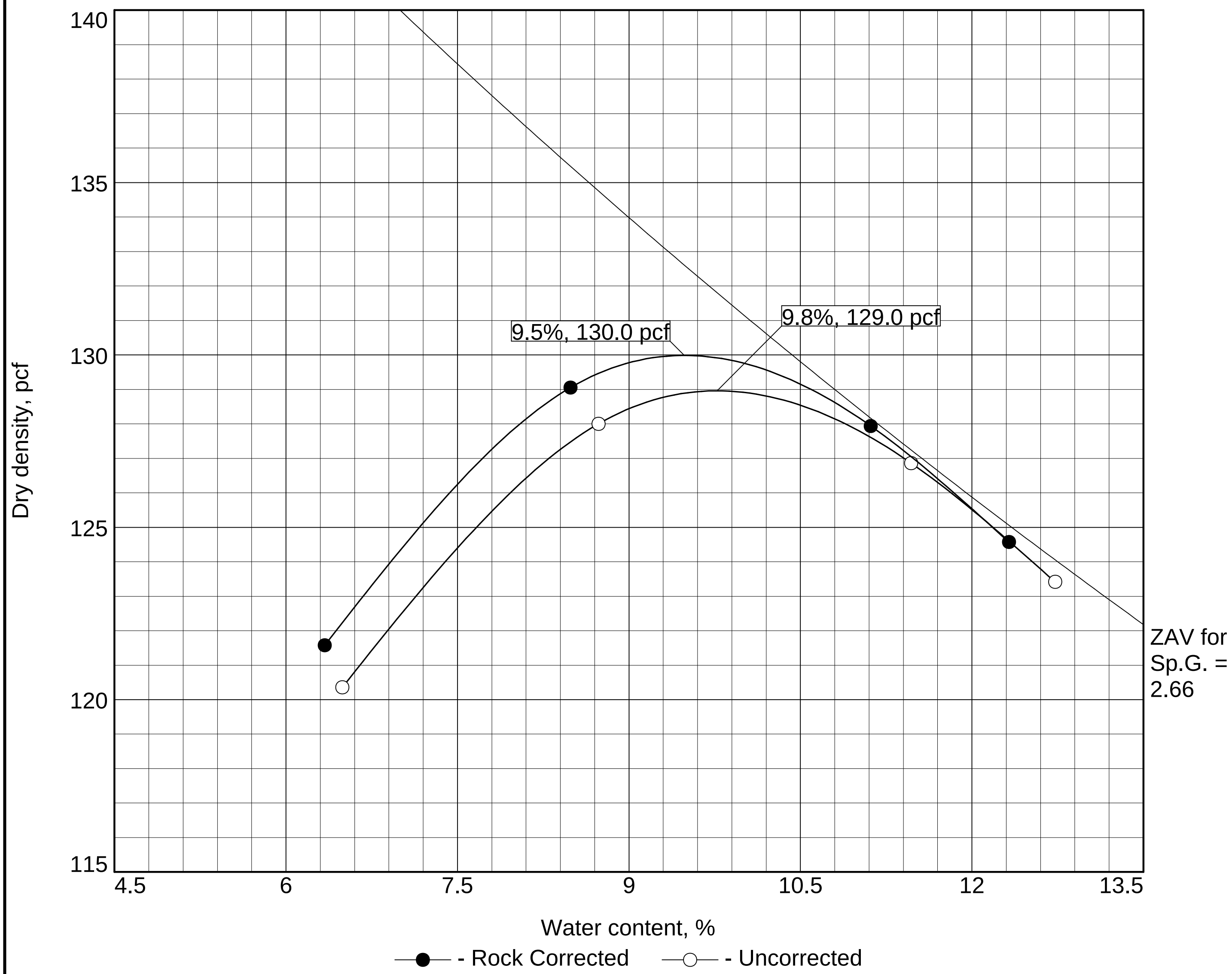
Remarks:

Blackburn Consulting

W. Sacramento, CA

Figure

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified, manual rammer, dry prep method
ASTM D 4718-87 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
3.0-6.0'	SC			2.66			4	29

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 130.0 pcf	129.0 pcf	CLAYEY SAND, reddish brown
Optimum moisture = 9.5 %	9.8 %	

Project No. 3228.X **Client:** Stantec - Rocklin
Project: LWWTRF Expansion Phase 1&2

○ **Source of Sample:** TP8 **Sample Number:** Bulk B

Remarks:

Blackburn Consulting

W. Sacramento, CA

Figure

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility

Phase 1 Expansion

Tertiary Storage Basin No. 3

Placer County, CA

APPENDIX C

Important Information About

This Geotechnical Engineering Report,

Geoprofessional Business Association

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
Maturation Pond Pump Station
Placer County, CA

Prepared by:

BLACKBURN CONSULTING
11521 Blocker Drive, Suite 110
Auburn, CA 95603
(530) 887-1494

April 2018

Prepared for:

Stantec
3875 Atherton Road
Rocklin, CA 95765

Auburn Office:
11521 Blocker Drive, Suite 110 ▪ Auburn, CA 95603
(530) 887-1494 ▪ Fax (530) 887-1495

blackburn consulting

Fresno Office: (559) 438-8411
West Sacramento Office: (916) 375-8706

Geotechnical ▪ Geo-Environmental ▪ Construction Services ▪ Forensics

File No. 3228.X
April 10, 2018

Mr. Gabe Aronow, P.E.
Stantec
3875 Atherton Road
Rocklin CA 95765

Subject: **GEOTECHNICAL DESIGN REPORT**
Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
Maturation Pump Station
Placer County, California

Dear Mr. Aronow:

Blackburn Consulting (BCI) is pleased to submit this Geotechnical Design Report for the Lincoln Wastewater Treatment and Reclamation Facility Phase 1 and Phase 2 Expansion Project, Maturation Pump Station located in Placer County, California. BCI prepared this report in accordance with our November 22, 2017 amendment.

This report presents geotechnical and geologic data, and provides recommendations to design and construct the new facilities.

Please call us if you have questions or require additional information.

Sincerely,

BLACKBURN CONSULTING

PROFESSIONAL GEOLOGIST ROBERT C. PICKARD No. 2508 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER THOMAS W. BLACKBURN No. 2311 Exp. GEOTECHNICAL STATE OF CALIFORNIA

Rob Pickard, P.G., C.E.G
Project Engineering Geologist

Thomas W. Blackburn, G.E., P.E.
Senior Principal

GEOTECHNICAL DESIGN REPORT
Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
Maturation Pond Pump Station
Placer County, CA

TABLE OF CONTENTS

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
Maturation Pond Pump Station
Placer County, CA

FIGURES

Figure 1: Vicinity Map
Figure 2: Site Map

APPENDIX A

Boring Logs (LWWTRF- 7)

Legend of Boring Logs

APPENDIX B

Laboratory Test Results

APPENDIX C

Important Information About This Geotechnical Engineering Report, Geoprofessional Business Association

1 INTRODUCTION

1.1 Purpose

Blackburn Consulting (BCI) prepared this Geotechnical Design Report for an expansion to the City of Lincoln Wastewater Treatment and Reclamation Facility (LWWTRF) located in Placer County, California. This report presents geotechnical and geologic data and provides recommendations to design and construct the new maturation pond pump station included in the Phase 1 and Phase 2 Expansion Project.

We are aware of the following geotechnical investigations on this site:

- 8/30/99 "Remote Storage Basins, East of Fiddyment Road, Placer County, California" by Carlton Engineering.
- 3/5/2001 "Geotechnical Investigation Report" by Kleinfelder.
- 1/31/2002 "Updated Geotechnical Investigation Report" by Kleinfelder.
- BCI, April 2013, Geotechnical Design Report, Mid-Western Placer Regional Sewer Project.
- BCI, November 2017, Geotechnical Design Report, Lincoln Wastewater Treatment and Reclamation Facility Phase 1 and 2 Expansion Project, WWTP Improvements.
- BCI, February 2018, Geotechnical Design Report, Lincoln Wastewater Treatment and Reclamation Facility Phase 1 Expansion, Tertiary Storage Basin No. 3 Project.

BCI prepared this report for Stantec to use during design and construction of the proposed improvements. Do not rely upon this report for different locations or improvements without the written consent of BCI.

1.2 Scope of Services

To prepare this report, BCI:

- Discussed the pump station improvements with Stantec
- Reviewed published geologic mapping, geotechnical information previously obtained for the project, and available geotechnical reports for existing facilities
- Performed a field investigation and laboratory analyses
- Performed engineering analysis and calculations

1.3 Site Location and Project Description

The LWWTRF project is located in an unincorporated area of Placer County. Figure 1 shows the project location.

As part of the LWWTRF Phase 1 and 2 Expansion Project a pump station, flow meter vault, and associated piping is proposed on the east levee between the existing north (unlined) and south (lined) maturation ponds. The project will also widen the levee crest in the area of the pump station by approximately 6 feet. The levee is approximately 12 feet high with, a crest elevation of approximately 116.5 feet. The new pump station will be constructed south of the existing pump station. We show the existing facilities, site topography, and proposed improvements on Figure 2.

2 GEOLOGIC CONDITIONS

2.1 General Geology

Our site work and published geologic mapping[1] show the site is underlain by Quaternary deposits of the Riverbank Formation. Our borings confirm that the levee fill is underlain by interbedded clays and sands.

The Riverbank Formation is an alluvial deposit typically composed of interbedded medium dense to dense sands, often cemented, and stiff to hard silts and clays. Bedding is typically horizontal, lenticular, and discontinuous. These sediments were deposited in the Late Pleistocene age (deposited over 150,000 years ago).

2.2 Faulting

The Fault Activity Map of California[2] does not identify Historic or Holocene age faults (displacement within the last 11,700 years) within or adjacent to the project site. The nearest mapped fault is the Cleveland Hill Fault located approximately 40 miles north of the site.

[1] Helley, E.J. and Harwood, D.S., 1985, Geologic Map of the Late Cenozoic Deposits of the Sacramento Valley and Northern Sierra Foothills: U.S. Geological Survey, Map MF-1790.
[2] Jennings, Charles W., and Bryant, William A., 2010 Fault Activity Map of California: California Geological Survey, Geologic Data Map No. 6.

3 FIELDWORK AND LABORATORY TESTS

3.1 Exploratory Borings

To characterize the subsurface conditions, BCI drilled, logged, and sampled one boring (B7) on December 8, 2018. The boring was drilled to a depth of 31.5 feet (elevation 85 feet) below the top of the existing levee. Figure 2 shows the approximate boring location. We include the boring log in Appendix A.

We located B-7 using geographic features shown on the project topographic mapping. We did not survey the exploration points.

Our subcontractor, Taber Drilling, drilled the boring using 4-inch solid-stem auger techniques. We obtained soil samples at various intervals using a 3.0-inch O.D. Modified California (MC) sampler (equipped with 2.4-inch diameter brass liners), driven with an automatic hammer, weighing 140-pounds and falling approximately 30 inches.

Ryan Schimdt, logged the borings and retrieved samples for laboratory testing. We used plastic caps to seal and label the 2.4-inch diameter, 6-inch long brass tubes retrieved from MC sampling. We also retrieved bulk soil samples from auger cuttings at varied depths, placed this material in large cloth bags, and labeled them for laboratory identification.

During our field exploration, we performed field strength estimates with a pocket penetrometer on select cohesive and/or cemented soil samples. We note the results of field tests on the boring logs.

3.2 Laboratory Testing

We completed the following laboratory tests on representative soil samples from our exploratory borings:

- Moisture content and unit weight for soil classification and in-place soil characteristics
- Expansion index for soil expansion potential
- Unconsolidated undrained triaxial test for strength characteristics
- Maximum dry density for compaction characteristics
- Soil corrosivity (pH, minimum resistivity, chlorides and sulfates) performed by Sunland Analytical Laboratories for soil corrosion characteristics

We attach a laboratory summary sheet and laboratory test results in Appendix B and show test results on the boring logs.

4 SUBSURFACE FINDINGS

4.1 Soil Conditions

We encountered the following soil profile in our boring:

- Stiff to very stiff lean clays, and clayey sands (interpreted to be levee fill) to depths of approximately 6 to 14 feet below ground surface (bgs). Pocket penetrometer tests range from 1.5 to 3.5 tons per square foot (tsf) and unconsolidated undrained triaxial strength of 1433 pounds per square foot (psf).
- Very stiff lean clays at depths of approximately 14 to 23 feet bgs (interpreted to be native soils). Pocket penetrometer tests of 3.5 to 3.75 tsf.
- Very dense and well graded sand at depths of approximately 23 to 28 feet bgs.
- Hard lean clay to the maximum depth explored (31.5 feet bgs). Pocket penetrometer test of 4.5 tsf.

Refer to the boring log (Appendix A) for more specific subsurface conditions.

4.2 Groundwater

We did not encounter groundwater in our boring. Groundwater has previously been recorded at shallower depths than what is shown above. Kleinfelder[3] recorded groundwater in their borings at depths ranging from 9.5 to 18 feet bgs (approximate elevations of 94.5 feet to 86 feet) in January 2001. It is not unusual to encounter channel sand lenses which can contain perched groundwater at varied depths within the Riverbank Formation. We also reviewed the Western Placer County Water Supply Appraisal[4], which shows regional groundwater elevations near 50 ft. Assume the highest groundwater elevation observed in the general area which is at an approximate elevation of 99 feet[5].

For project design, we assume that our boring reflects normal water levels within the levee. However, we assume higher water levels in the Maturation Ponds will affect water levels at the pump station. Water levels will likely be higher when construction of the future Maturation Pond No. 3 (to be located west of the pump station) is completed. We assume that long term water levels in the levee will match the water levels in the adjacent maturation ponds. Use a design water level based on the anticipated high water level.

[3] Kleinfelder, 2002, Updated Geotechnical Investigation Report, Proposed Lincoln Wastewater Treatment Plant, Fiddyment Road, Placer County, California; consultant's report to Del Webb California Corporation
[4] Boyle Engineering, Western Placer County Water Supply Appraisal, Groundwater Elevations, Spring 1987.
[5] Kleinfelder, 2002, Updated Geotechnical Investigation Report, Proposed Lincoln Wastewater Treatment Plant, Fiddyment Road, Placer County, California; consultant's report to Del Webb California Corporation

5 CONCLUSIONS AND RECOMMENDATIONS

The site will be suitable for the planned facilities when constructed in accordance with the project plans, industry standards, and our geotechnical recommendations. Some of the more significant site limitations include possible shallow groundwater that may require dewatering for some structure installations.

5.1 Geologic Hazards

- Faulting—The potential for surface rupture or creep due to faulting at the site is very low. The Fault Activity Map of California[6] and the Geologic Map of the Sacramento Quadrangle[7] does not identify Historic or Holocene age faults (displacement within the last 11,700 years) within or immediately adjacent to the site. The site does not lie within or adjacent to an Alquist–Priolo Earthquake Fault Zone[8].
- Ground Shaking—The USGS, Earthquake Hazards Program, Seismic Design Maps (https://earthquake.usgs.gov/designmaps/us/application.php) indicate that for the design seismic event, a peak horizontal ground acceleration (PGA) of approximately 0.172g could be expected.
- Liquefaction—Our investigation shows a soil profile that consists of stiff to hard clays and medium dense to dense silty and clayey sands that are not liquefiable. Therefore, the potential for damaging liquefaction at the site is very low.
- Landslides and Slope Stability—Due to the relatively low topographic relief and existing slope gradients we do not expect landslides or natural slope failure.
- Seismically Induced Settlement—During a seismic event, ground shaking can cause densification of granular soil that can result in settlement of the ground surface. Considering the cohesive soils and medium dense soils observed in the borings, we consider the potential for significant seismically induced settlement to be very low.

5.1 Seismic Design

The project site is underlain by dense/very stiff to hard soils which is considered as Site Class C in the California Building Code (CBC).[9]

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[6] Jennings, Charles W., and Bryant, William A., 2010 Fault Activity Map of California: California Geological Survey, Geologic Data Map No. 6.
[7] Wagner, D.L., et al, 1981, Geologic map of the Sacramento quadrangle, California, 1: 250,000: California Division of Mines and Geology, Regional Geologic Map 1A, scale 1: 250,000.
[8] Bryant, W.A., and Hart, E.W., 2007 (Interim Revision), Fault-Rupture Hazard Zones in California: California Department of Conservation, Division of Mines and Geology, Special Publication 42.
[9] California Building Code, 2016, California Code of Regulations, Title 24, Part 2 (Volume 2); published by International Conference of Building Officials and the California Building Standards Commission.

For seismic design of plant components, use the values in Table 1:

TABLE 1

CBC Seismic Design Parameters[10] (Site Class C)	
S_s – Acceleration Parameter	0.516 g
S_1 – Acceleration Parameter	0.254g
F_a – Site Coefficient	1.1954
F_v – Site Coefficient	1.546
S_{MS} – MCE* Spectral Response Acceleration, Short Period	0.616 g
S_{M1} – MCE* Spectral Response Acceleration, 1-Second Period	0.393 g
S_{DS} – 5% Damped Design Spectral Response Acceleration, Short Period	0.411 g
S_{D1} – 5% Damped Design Spectral Response Acceleration, 1-Second	0.262 g
T_L – Long Period Design Period**	12 seconds
PGA – Peak Ground Acceleration	0.172 g
PGA_M – Site Modified Peak Ground Acceleration	0.207 g

* Maximum Considered Earthquake
** Figure 22-12, ASCE 7-10

5.2 General Grading Recommendations

5.2.1 *Excavation Conditions*

Based on the soil conditions and drilling performance, excavation is possible with conventional equipment (common earthmoving equipment and large backhoe/excavator). The fine-grained and hard soil conditions can create slow excavation conditions.

5.2.2 *Site Clearing*

Prior to trenching or making any cuts and fills, remove all debris, and brush including the root system and strip surface vegetation to a depth of 4 inches below the surface. Excavations resulting from brush, and debris removal should be deepened and widened to provide access to self-propelled compaction equipment. Remove strippings from the site or use as landscape soil in designated areas.

[10] California Building Code, 2016, California Code of Regulations, Title 24, Part 2 (Volume 2); published by International Conference of Building Officials and the California Building Standards Commission.

5.2.3 *Original Ground and Subgrade Preparation*

After clearing process and compact the exposed soil in at-grade, cut, and fill areas as follows:

- Scarify the exposed soil to a depth of approximately 8 inches.
- Moisture condition subgrade to within 3% of the optimum moisture content.
- Compact the subgrade soil to a minimum 90% relative compaction based on ASTM D1557

Where fill is placed on sloping ground, blade back slopes horizontally during placement of embankment fill to create a stepped (or benched) fill surface (such that a uniform, sloping fill surface is avoided). Benching must remove loose surficial soils and result in stepped benches, generally one to two feet in height and depth into the existing slope. The lower bench should be sloped a minimum of 2% into the slope. Where benching will interfere with existing structures, utilities, or vegetation, BCI can review modifications on a case-by-case basis.

5.2.4 *General Fill Placement and Compaction*

General fill may consist of on-site soil. Fill should be free of debris and concentrations of vegetation.

Import fill for use pump station and levee improvements should meet the following criteria:

- 100 % passing the 3-inch sieve
- 90% to 100% passing the 2-inch sieve
- 75% to 100% passing the No. 4 sieve
- 20-60% passing the No. 200 sieve
- Liquid Limit ≤ 45
- Plasticity Index ≥ 8 and ≤ 20
- Shall not contain organics, debris or other deleterious material
- Approval from BCI prior to placement

Place fill in maximum 8-inch thick loose lifts, moisture condition 1% to 2% above optimum, and compact to a minimum of 90% relative compaction based on ASTM D 1557 test procedure. Compact fill using a sheepsfoot or padded drum type roller.

Construct fill slopes no steeper than 2(H):1(V). To achieve adequate compaction on the face of fill slopes, over-build the slopes and then cut back to the design grade. Track-walking is not an adequate method to compact the face of slopes.

5.3 Dewatering

Dewatering may be required for installations greater than approximately 17 feet deep (elevation 99 feet, see Section 4.2). Significant groundwater inflow may occur at the pump station, particularly during winter and spring months.

Dewatering can consist of:

- Deep sumps within the excavation. Considering the presence of fine-grained soils and relatively flat lying bedding, sumps within the excavation are not likely to provide good drawdown.
- Well points. Well points will likely work better to cut off flow into the excavation and drawdown the water level over a larger area.

To facilitate work at the base of the excavation, groundwater should be drawn down at least 5 feet below the planned bottom of excavation. The need for dewatering can be reduced by planning excavations during the lowest anticipated seasonal water levels (expected during the late summer and fall months) and lowering the water level in the unlined maturation pond as much as possible.

5.4 Temporary Excavations

Temporary excavations will require sloping and/or shoring in accordance with Cal OSHA requirements. Based on our subsurface exploration and laboratory testing, preliminary excavation and shoring design may be based on Type B soil to planned excavation depth. For Type A soil conditions, temporary excavations may be sloped at 1(H):1(V).

Where groundwater is present or cohesionless/uncemented granular soils are encountered, Type C soil conditions will apply and a 1.5(H):1(V) slope gradient is required.

The impact of existing structures, traffic vibrations, actual soil conditions exposed in the open trenches, and other factors that may promote trench wall instability must be evaluated at the time of construction and trench sloping/shoring adjusted accordingly. Surcharge loads such as trench spoils, equipment, etc. should not be placed adjacent to an open excavation (within a distance of ½ the height of the trench). ***The above is guideline information only.*** The contractor is responsible for the safety of all excavations and should provide appropriate excavation sloping and shoring in accordance with current Cal OSHA requirements and observe conditions observed during construction for necessary modification and safety.

5.5 Foundation Design

5.5.1 *Below-Grade Foundations*

5.5.1.1 <u>Bearing Capacity</u>

The pump station is a below-grade structure and the net pressure exerted upon the subsurface will be similar to or less than the current load. Excavation for below-grade structures reduces the net pressure by removing soil that acts as a "preload" to the underlying soils, thus "unloading" the bearing materials before "loading" by placement of the structure.

Below grade structures will use mat type foundations for support. For structures at depths greater than 18 feet (approximate elevation 98.5 feet):

- Use a maximum net contact pressure for mat foundation of 2,000 psf.
- We expect settlement of mat foundations is expected to be less than 1 inch with differential settlement less than ½-inch across the pump station structure.
- Clean footing excavations of debris and loose soil prior to placing concrete.
- BCI must observe all footing excavations prior to reinforcement placement to verify competent bearing materials.
- For subgrade uniformity, Caltrans Class 2 aggregate baserock as underlayment (this is not geotechnically necessary provided a firm uniform subgrade is obtained). If an aggregate underlayment is used, place a minimum thickness of 6-inches and compact to a minimum of 95% relative compaction (per ASTM D 1557 test method).
- Crushed rock underlayment may also be used (and can benefit excavation dewatering). Underlay the crushed rock with a geotextile filter fabric (ie. Mirafi 140N) and compact the rock with at least 6 passes of a static roller.

If isolated spread footings or piers are required for column support, BCI can provide additional recommendations when the planned design and approximate loading is available.

5.5.1.2 <u>Structure Backfill</u>

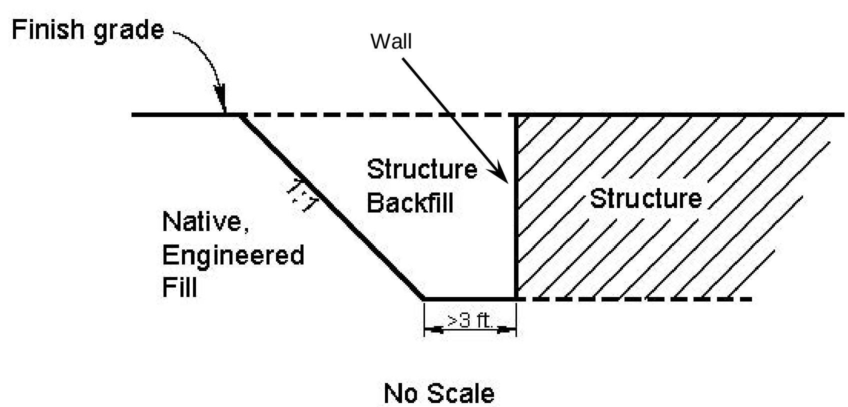
Levee fill consists predominately of lean clay and clayey sands. This material may used as backfill around the new pump station.

If imported fill is required use the specifications in Table 2 for structure backfill for all below-grade structures:

TABLE 2

Structure Backfill Requirements			
Gradation		**Test Procedures**	
Sieve Size	**Percent Passing**	**ASTM**	**Caltrans**
3 inch	100	D6913	202
¾ inch	70-100	D6913	202
No. 4	50-100	D6913	202
No. 200	20-60	D6913	202
Plasticity			
Plasticity Index	≥ 8 and ≤ 20	D4318	204
Organic Content			
Less than 3%		D2974	
Expansion Index			
Less than 20		D4829	

As shown below, the zone of placement for structure backfill should extend up from the base of the wall at a slope of 1(H):1(V) and at least 3 feet behind the wall.



- Moisture condition backfill to within 2% of optimum and place in maximum 8-inch thick, horizontal, loose lifts.
- Compact backfill to a minimum 92% relative compaction based on the ASTM D 1557 test method.

To minimize the residual lateral earth pressures on structure walls, restrict compaction equipment behind the walls (by load and distance from wall) so that wall design values are not exceeded. We recommend compaction within a horizontal distance equal to one-half of the wall height (to a maximum distance of 5 feet), be completed with hand-operated equipment (i.e., jumping jack).

To minimize the potential for significant settlement around deep walls, controlled low strength material (CLSM) can be used to backfill to the surface or to a manageable depth (e.g. 10 feet below grade).

5.5.1.3 Lateral Earth Pressures

The below grade structures will act as retaining structures. Walls will retain compacted select native soils and/or imported soils meeting the requirement for structure backfill. For evaluation of lateral earth pressures, use the undrained backfill with level ground conditions equivalent fluid weights (EFW) shown below in Table 3.

TABLE 3

LATERAL EARTH PRESSURES	
Condition	**Undrained Equivalent Fluid Weight (pcf)**
At-Rest	100
Active	86
Passive	270 (F.S. = 1)
Seismic (Active and At-Rest)	6

The above pressures assume structure backfill placed against the structure wall in accordance with our recommendations, and a saturated unit weight of approximately 133 pounds per cubic foot (pcf). Notify BCI if these assumptions are not valid so that we may assess the situation and provide additional recommendations, if necessary. Backfill with CLSM is an acceptable alternative.

For seismic loading, add the Seismic EFW to the at-rest or active EFW weight and apply the total force as a uniform load on the wall with a resultant located at 0.5H where H is the backfill height. We estimated the EFWs for seismic loading using the Mononobe-Okabe equation and a horizontal seismic acceleration coefficient, k_h, of approximately ½ the expected PGA. This k_h value assumes that the walls displace at least 1-inch during the design seismic event.

Surface loads (footings, storage, vehicle traffic) applied near the wall will increase the lateral pressure on the wall. A uniform surface load of 200 psf to 300 psf is often used to approximate construction traffic loading on walls. In general, if surface loads are closer to the edge of the retaining wall than three-fourths of the retained height, increase the design wall pressure by 0.5q over the area of the retaining wall. In this expression, q is the surface surcharge load in psf. This is a conservative procedure and lower design pressures may be applicable upon evaluation of individual surface loads and setback distances.

5.5.1.4 Buoyancy Resistance

We did not encounter groundwater in B7, however, as discussed in section 4.2, groundwater may occur at elevations as shallow as 99 feet. In undrained conditions, structures below approximate elevation 99 feet, may be subjected to an uplift load (buoyancy). The uplift force will be resisted by the weight of the structure and the weight of the backfill overlying foundation extensions (if any).

If Stantec designs foundation extensions, calculate the resistance against uplift due to the weight of the soil, use a backfill total unit weight of 120 pcf above groundwater and 57 pcf below groundwater, with a soil wedge extending up from foundation extensions at an angle of 30 degrees from vertical.

Frictional resistance from surrounding soils can be used to resist uplift as well. The frictional resistance will vary with depth but can be assumed as follows (apply a factor of safety of at least 2 to determine the allowable uplift resistance):

For structure backfill against a concrete structure:

- 24 psf per foot of depth where above the design groundwater level
- 13 psf per foot of depth when below the design groundwater level

For a vertical soil interface such as over a foundation extension:

- 38 psf per foot of depth where above the design groundwater level
- 21 psf per foot of depth when below the design groundwater level

5.5.1.5 Lateral Resistance

Lateral resistance for retaining structures can be achieved through friction and passive earth pressures. For design, use a coefficient of friction of 0.40 (below or above groundwater) at the base of the concrete footing and a passive earth pressure of 135 psf per foot of embedment depth. Passive earth pressures may be increased up to 270 psf per foot if lateral movements of up to 2% of the embedment depth can be tolerated. Limit passive earth pressures to a maximum of 2,000 psf (additional passive pressure can be evaluated for specific locations if necessary). Do not include the upper 1-foot of soil in passive resistance calculations. Where passive pressure or friction alone is used against sliding, use a minimum factor of safety of 1.5 for lateral stability (1.1 if seismic loading is included). Where both passive pressure and friction are used to resist sliding, use a minimum factor of safety of 2.0.

5.6 Minor Structures (Valve Vault)

Provided that the recommendations in this report are followed, minor structures (such as valve, vaults, etc.) may be founded on concrete mat or strip footings, or a compacted granular base (minimum of 6 inches of Class 2 baserock) if appropriate.

- Embed the foundations a minimum of 18 inches below the lowest adjacent prepared subgrade into firm native soil or compacted fill/backfill.
- Footings must be a minimum of 12 inches wide and sized not to exceed an allowable bearing capacity of 2,000 psf. The allowable bearing capacity may be increased by one-third if seismic and/or wind loads are included.
- If additional bearing capacity is required for specific minor structures, we can review and provide recommendations on a case-by-case basis.
- To resist lateral movement, use a coefficient of friction of 0.40 at the base of the foundation and a passive earth pressure of 270 psf (undrained condition) per foot of embedment depth up to a maximum of 2,000 psf. Ignore the upper one-foot of footing depth (below the lowest adjacent soil grade) in determination of the passive pressure. Both frictional resistance and passive earth pressure can be combined for lateral resistance; when combined, increase the safety factor against sliding from a minimum of 1.5 to 2.0.

If necessary for evaluation of lateral loading on shallow vaults, use an At-Rest equivalent fluid weight of 60 pcf for the drained condition and 100 pcf for undrained. The drained condition assumes groundwater does not accumulate; the undrained condition would be applied below an assumed groundwater level.

We based these values on foundations bearing on compacted levee soils and soil meeting the embankment fill requirements compacted against vault walls.

5.7 Soil Corrosivity

Our subcontractor, Sunland Analytical, tested a soil sample from our boring for corrosion characteristics (pH, resistivity, chlorides, and sulfates). The test shows:

- pH = 7.31
- Minimum Resistivity = 1,820
- Chloride = 8.0 ppm
- Sulfate = 23.9 ppm

American Concrete Institute (ACI) 318 Table 4.3.1 provides guidance on concrete exposed to sulfate. Results of laboratory testing indicate a negligible sulfate exposure for the representative soil samples.

Caltrans considers a site to be corrosive if one or more of the following conditions exist for the representative soil samples taken at the site:

- Chloride concentrations greater than or equal to 500 parts per million (ppm),
- Sulfate concentration is greater than or equal to 2000 ppm, or
- pH is 5.5 or less.

Based on these test results, the site would be considered non-corrosive. However, the resistivity values and the presence of the fine-grained soils suggest the soil may be corrosive to metals. We recommend that a corrosion engineer review these results and provide corrosion mitigation recommendations.

5.8 Inlet/Outlet Pipe Installation

We expect adequate foundation support for pipes placed in native soil and compacted levee fill and that settlement will be negligible following proper placement and backfill. We expect trench excavations to be relatively stable. For preliminary consideration, use a Type B soil classification (Federal Register, OSHA, 29 CFR Part 1926) for temporary trench sloping and/or shoring design. Excavations may encounter clayey or clean sands, or groundwater, in which case sloping/shoring will need to be modified for a Type C soil classification. Final sloping/shoring based on actual conditions is the responsibility of the contractor.

For pipe beneath the existing embankment, construct in accordance with the following:

- Best option: Use controlled, low strength material (CLSM) to backfill and encapsulate the pipe (which also allows a narrower trench).
- Place the CLSM a minimum of 2 feet above the pipe if embankment fill is to be placed as intermediate trench backfill.

Or:

- Excavate the trench to a depth of approximately 2 feet below the bottom of the pipe and at least 4 feet wider than the pipe to encapsulate the pipe with an “impermeable” zone of engineered fill around the pipe.
- Selectively stockpile material so the contractor can be reuse it as backfill.
- After the contractor excavates the trench, backfill it to the pipe invert elevation. Compact the backfill with mechanical compactors to a minimum of 90% percent relative compaction near optimum moisture content.
- Bring backfill up evenly on both sides of the pipe to avoid unequal side loads that could fail or move the pipe. Take special care in the vicinity of any protrusions such as joint collars to achieve proper compaction.

6 RISK MANAGEMENT

Our experience and that of our profession clearly indicates that the risks of costly design, construction, and maintenance problems can be significantly lowered by retaining the geotechnical engineer of record to provide additional services during design and construction.

For this project, we recommend that the project owner retain us to:

- Review and provide comments on the civil plans and specifications prior to construction.
- Monitor construction to check and document our report assumptions. At a minimum, BCI should observe foundation excavations, approve backfill, test backfill compaction, observe and test placement and compaction of fill for structures.
- Update this report if design changes occur, 2 years or more lapses between this report and construction, and/or site conditions have changed.

If we are not retained to perform the above applicable services, we are not responsible for any other party's interpretation of our report, and subsequent addendums, letters, and discussions.

7 LIMITATIONS

BCI performed services in accordance with generally accepted geotechnical engineering principles and practices currently used in this area. Where referenced, we used ASTM and California Test Method standards as a general (not strict) guideline only. Do not use or rely upon this report for different locations or improvements without the written consent of BCI.

We do not warranty our services.

BCI based this report on the current site conditions. We assume our boring and groundwater conditions are representative of the subsurface conditions throughout the site. Conditions at locations other than our exploration could be different.

Appendix A shows logs of our exploration. The lines designating the interface between soil types are approximate. The transition between material types may be abrupt or gradual. We based our recommendations on the final log, which represents our interpretation of the field log and general knowledge of the site and geological conditions. We based our boring log descriptions on our field logging, geologic mapping, and laboratory testing.

The groundwater elevations discussed in this report represent the groundwater elevation during the time of our subsurface exploration, at the specific exploration location, and groundwater observed by others. The groundwater table may be lower or higher in the future.

Modern design and construction are complex, with many regulatory sources/restrictions, involved parties, construction alternatives, etc. It is common to experience changes and delays. The owner should set aside a reasonable contingency fund based on complexities and cost estimates to cover changes and delays.

Appendix C shows GBA guidelines for how to use this report.

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility

Phase 1 and Phase 2 Expansion Project
Maturation Pond Pump Station
Placer County, CA

FIGURES

N

SCALE 1" = 8,000'

4/10/2018 3228.x Fig1 Maturation Pump Station.dwg

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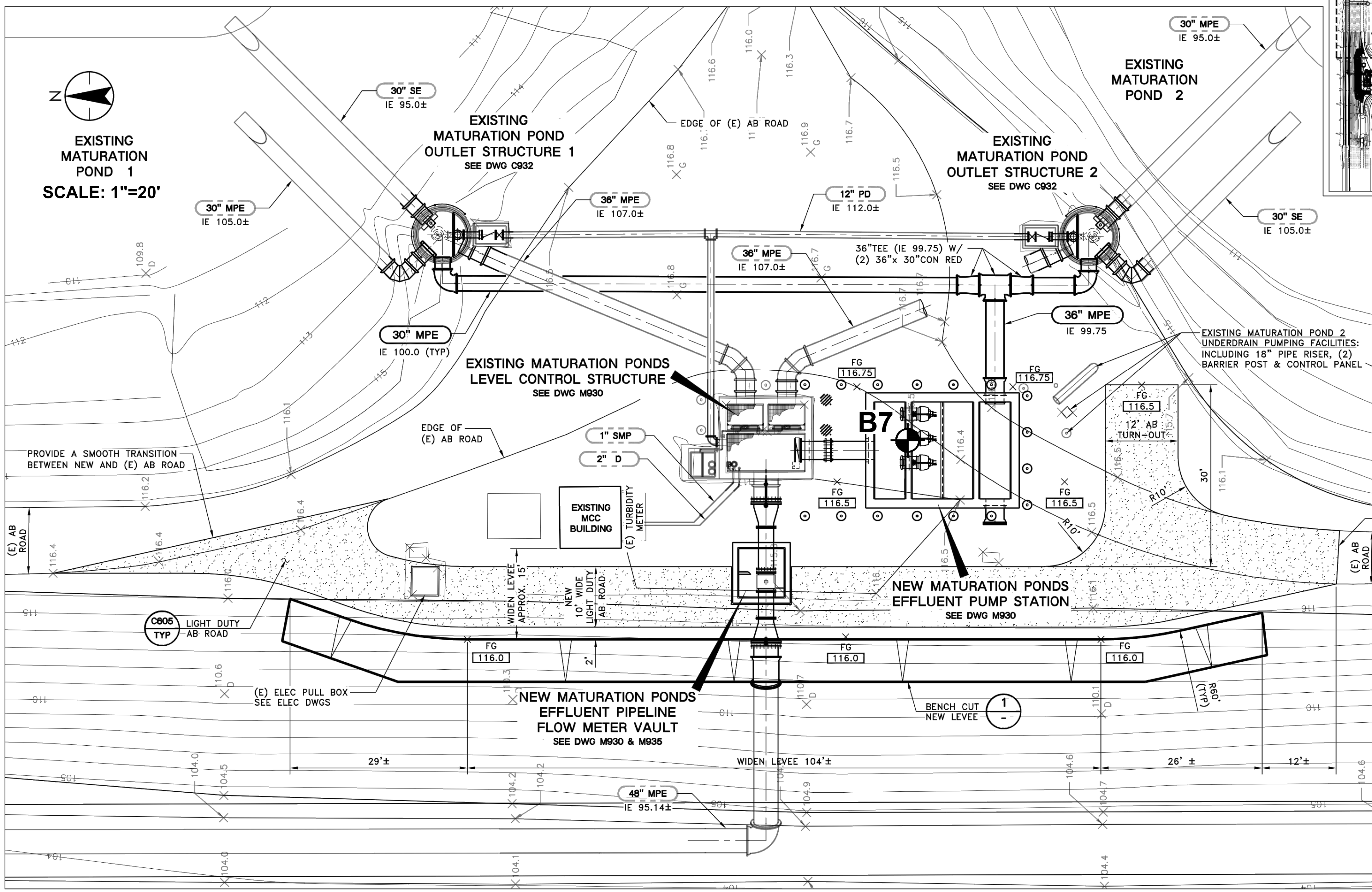
VICINITY MAP

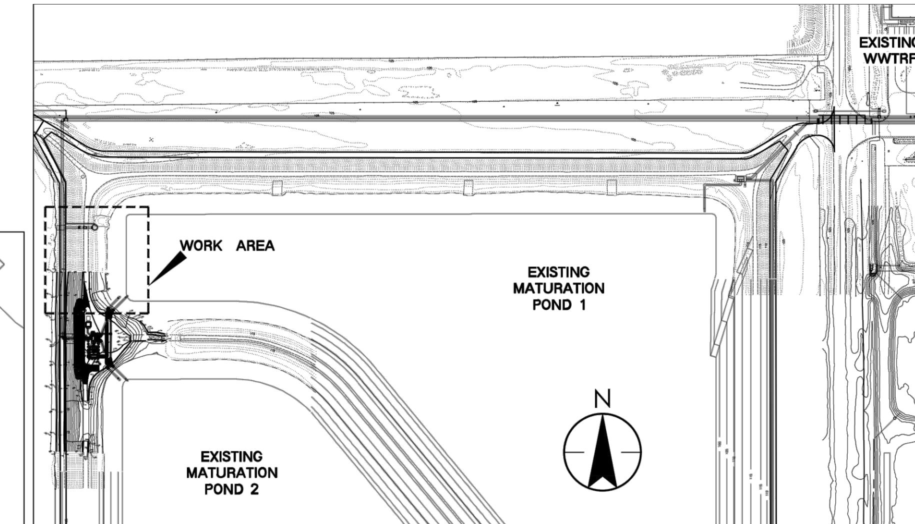
Lincoln Wastewater Treatment and Reclamation Facility Phase 1 Expansion
Maturation Pump Station
Placer County, California

File No. 3228.x

April 2018

Figure 1





MATURATION PONDS KEY MAP

SCALE: 1" = 200'

LEGEND

Approximate Boring Location

SOURCE: 95% Submittal Plan, Existing Maturation Ponds Modification Plan, Drawing No. C930 by Stantec. Plot date 12/21/17.

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SITE MAP

Lincoln Wastewater Treatment and Reclamation Facility Phase 1 Expansion
Maturation Pump Station
Placer County, California

File No. 3228.x

April 2018

Figure 2

4/10/2018 3228.x Fig2 Maturation Pump Station.dwg

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility

Phase 1 and Phase 2 Expansion Project

Maturation Pond Pump Station

Placer County, CA

APPENDIX A

Boring Logs (LWWTRF- 7)

Legend of Boring Logs

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
RMS	**12-8-17**	**12-8-17**	**38.859181° / -121.354851°**	**B7**

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
Taber		**116.5 ft**
DRILLING METHOD	DRILL RIG	BOREHOLE DIAMETER
Solid-Stem Auger	**Diedrich D120**	**4 in**
SAMPLER TYPE(S) AND SIZE(S) (ID)	HAMMER TYPE	HAMMER EFFICIENCY, ERi
2.4" CAMOD	**Safety semi-automatic drop (140#/ 30")**	**Approx. 80%**
BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS — DURING DRILLING: **None**; AFTER DRILLING (DATE): **None**	TOTAL DEPTH OF BORING
Backfill with Tremie Grout		**31.5 ft**

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION/REMARKS	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests
	0	Lean CLAY with SAND (CL); Stiff; Dark Brown; Moist; Fine SAND; Medium Plasticity; Fill											CP, CR, EI
	1												
114.50	2		1	3 4 8	12	20	108					PP = 2.0	
	3												
112.50	4												
	5	20% Well Graded SAND	2	7 5 7	12						UU = 1432.9	PP = 3.5	
110.50	6												
	7												
108.50	8												
	9												
106.50	10	Soft	3	2 4 7	11							PP = 1.5	
	11	CLAYEY SAND (SC); Medium Dense; Reddish Brown; Moist; Well Graded SAND; 20-30% CLAY; Fill											
104.50	12												
	13												
102.50	14	Lean CLAY (CL); Very Stiff; Dark Brown; Moist; Medium Plasticity											
	15		4	10 12 15	27	17	113					PP = 3.75	
100.50	16												
	17												
98.50	18	SANDY Lean CLAY (CL); Very Stiff; Brown; Moist; 35% Fine SAND; Medium to Low Plasticity											
	19												
96.50	20		5	8 12 15	27							PP = 3.5	
	21												
94.50	22												
	23	Well Graded SAND with CLAY (SW-SC); Very Dense; Reddish Brown; Moist; 10% CLAY; Traces of GRAVEL											
92.50	24												
	25												

(continued)

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PROJECT NAME		FILE NO.	HOLE ID
Lincoln WWTRF TSB No. 3		**3228.X**	**B7**
COUNTY: **PLA**	ROUTE	POSTMILE	
CLIENT: **Stantec**			
PREPARED BY: **RMS**	CHECKED BY: **RCP**	SHEET: **1 of 2**	

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (psf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	25				6	25 46 50/4"	96/10										
90.50	26																
	27																
88.50	28		Lean CLAY with SAND (CL); Hard; Brown; Moist; 20-30% Fine SAND; Medium to Low Plasticity														
	29																
86.50	30				7	14 18 24	42							PP = 4.5			
	31																
84.50	32		Bottom of borehole at 31.5 ft bgs Backfill with Tremie Grout No Groundwater Encountered Bulk A: 0-5 ft Bulk B: 5-10 ft														
	33																
82.50	34																
	35																
80.50	36																
	37																
78.50	38																
	39																
76.50	40																
	41																
74.50	42																
	43																
72.50	44																
	45																
70.50	46																
	47																
68.50	48																
	49																
66.50	50																
	51																
64.50	52																
	53																
62.50	54																
	55																

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PROJECT NAME		FILE NO.	HOLE ID
Lincoln WWTRF TSB No. 3		3228.X	B7
COUNTY: PLA	ROUTE	POSTMILE	
CLIENT: Stantec			
PREPARED BY: RMS	CHECKED BY: RCP	SHEET: 2 of 2	

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (CTM 216 - 06)
CR	Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content (ASTM D 2974-07)
P	Permeability (CTM 220 - 05)
PA	Particle Size Analysis (ASTM D 422-63 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301 - 00)
SE	Sand Equivalent (CTM 217 - 99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96 [2004])

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- 2.5" ID Sampler
- 2" ID Sampler
- Shelby Tube
- Piston Sampler
- NX Rock Core
- HQ Rock Core
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (short-term)
- Static Water Level Reading (long-term)

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BORING RECORD LEGEND

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60} - Value (blows / foot)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Descriptor	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Descriptor	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE: This legend sheet provides descriptors and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010), Section 2, for tables of additional soil description components and discussion of soil description and identification.

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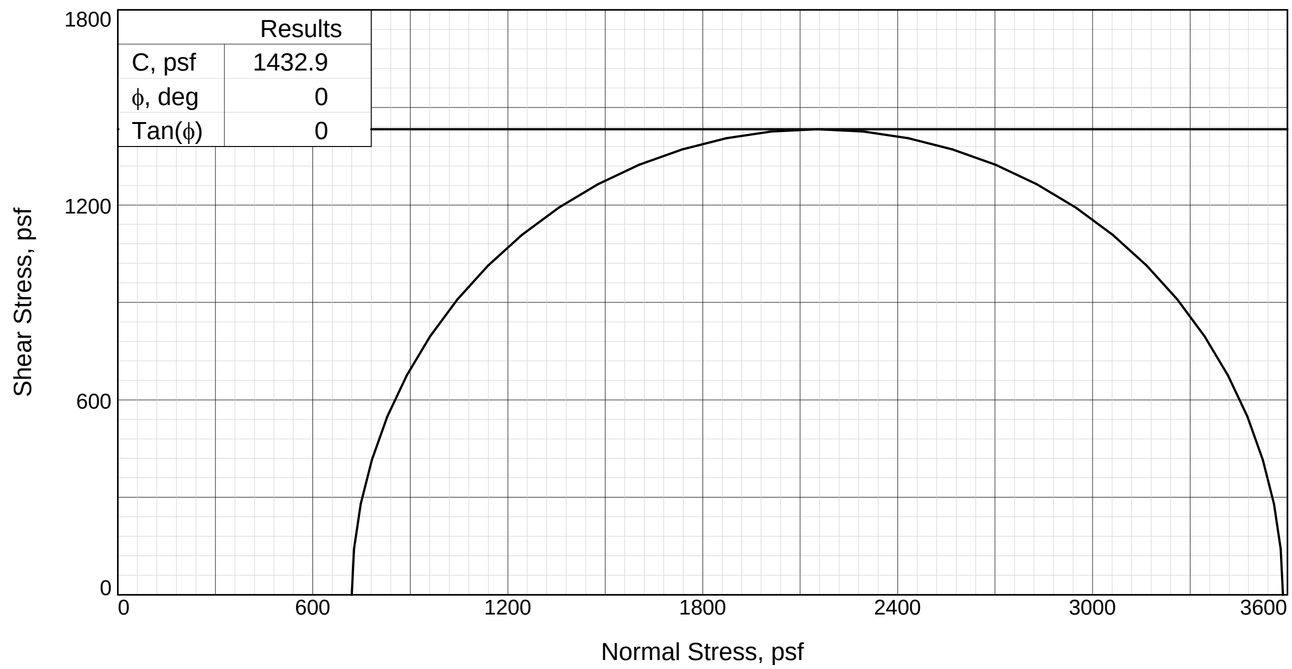
BORING RECORD LEGEND

GEOTECHNICAL DESIGN REPORT

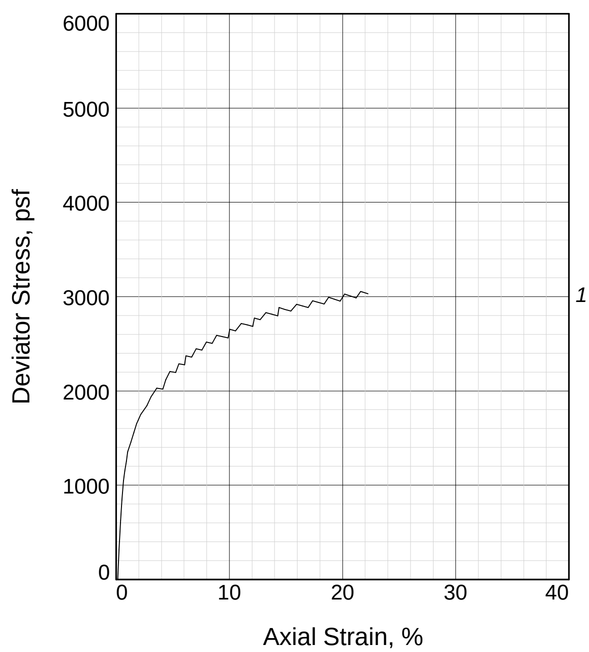
Lincoln Wastewater Treatment and Reclamation Facility
Phase 1 and Phase 2 Expansion Project
Maturation Pond Pump Station
Placer County, CA

APPENDIX B

Laboratory Test Results



Results	
C, psf	1432.9
ϕ, deg	0
Tan(ϕ)	0



	Sample No.	1
Initial	Water Content, %	18.2
	Dry Density, pcf	106.2
	Saturation, %	83.4
	Void Ratio	0.5877
	Diameter, in.	2.400
	Height, in.	4.439
At Test	Water Content, %	18.2
	Dry Density, pcf	106.2
	Saturation, %	83.4
	Void Ratio	0.5877
	Diameter, in.	2.400
	Height, in.	4.439
	Strain rate, in./min.	0.044
	Back Pressure, psf	0.0
	Cell Pressure, psf	720.0
	Fail. Stress, psf	2865.7
	Strain, %	14.9
	Ult. Stress, psf	
	Strain, %	
	$\pm_1$ Failure, psf	3585.7
	$\pm_3$ Failure, psf	720.0

Type of Test:
Unconsolidated Undrained

Sample Type: 2.4" Mod Cal

Description: SANDY lean CLAY with GRAVEL, yellowish brown

Assumed Specific Gravity= 2.70

Remarks:

Figure ________

Client: Stantec - Rocklin

Project: LWWTRF Expansion Phase 1&2

Source of Sample: B7

Sample Number: 2C

Proj. No.: 3228.X **Date Sampled:** 1/12/18

TRIAXIAL SHEAR TEST REPORT

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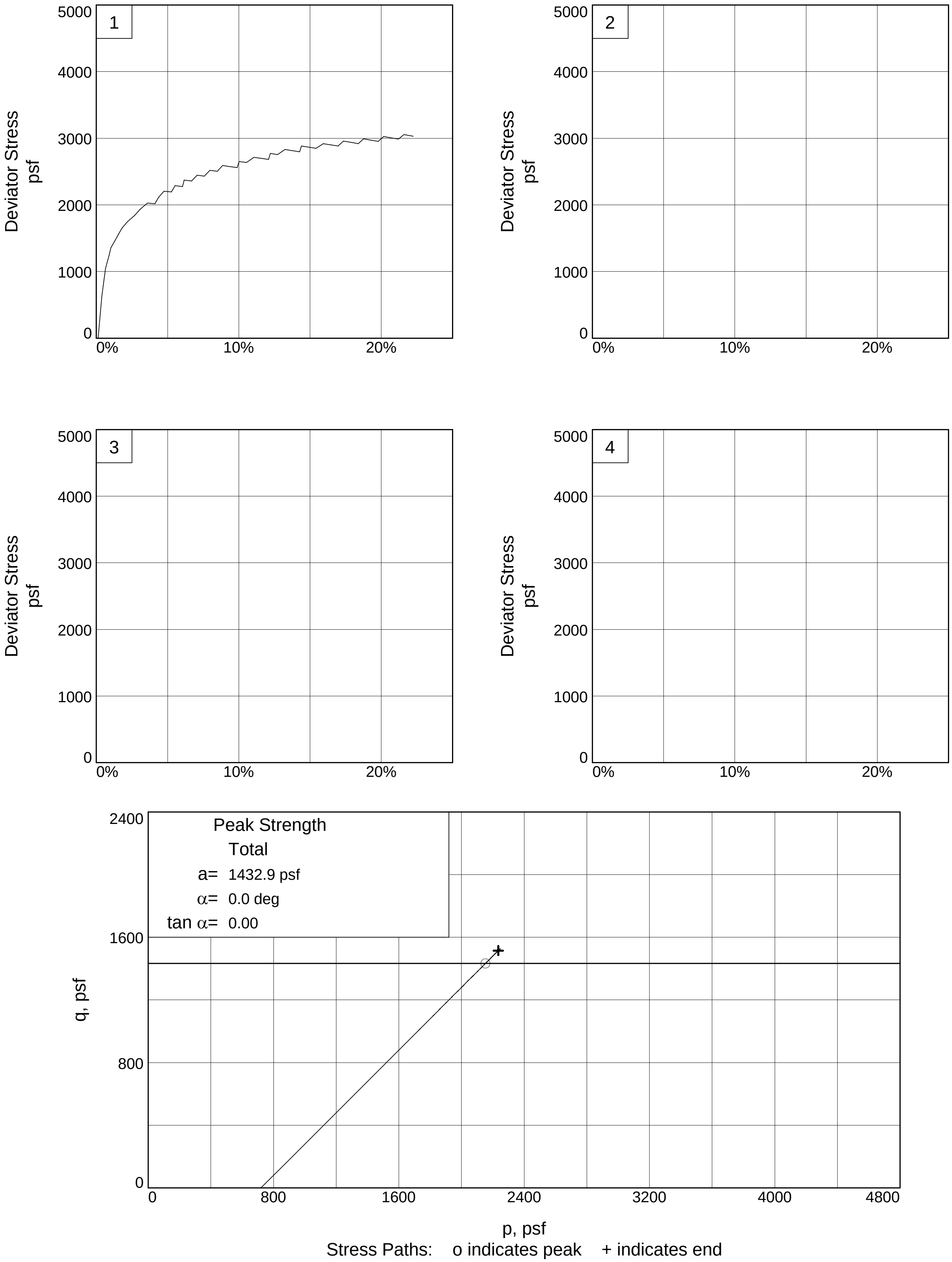
W. Sacramento, CA



Client: Stantec - Rocklin
Project: LWWTRF Expansion Phase 1&2
Source of Sample: B7 **Sample Number:** 2C
Project No.: 3228.X **Figure** ________

Blackburn Consulting

Project Name: LWWTRF
Project No: 3228.X
Sample No: B7 Bulks A&B
Depth 0.0-10.0'
Date: 1/30/2018
Sample Description: CLAYEY SAND, dark yellowish brown

EXPANSION INDEX TEST
(ASTM D4829)

Test Data Summary

Retained #4 (%)	0.0%
Initial Moisture (%)	12.0
Final Moisture (%)	22.4
Percent Saturation (%)	51.6
Initial Dry Density (pcf)	103.5
Final Dry Density (pcf)	101.1
Expansion Index	13

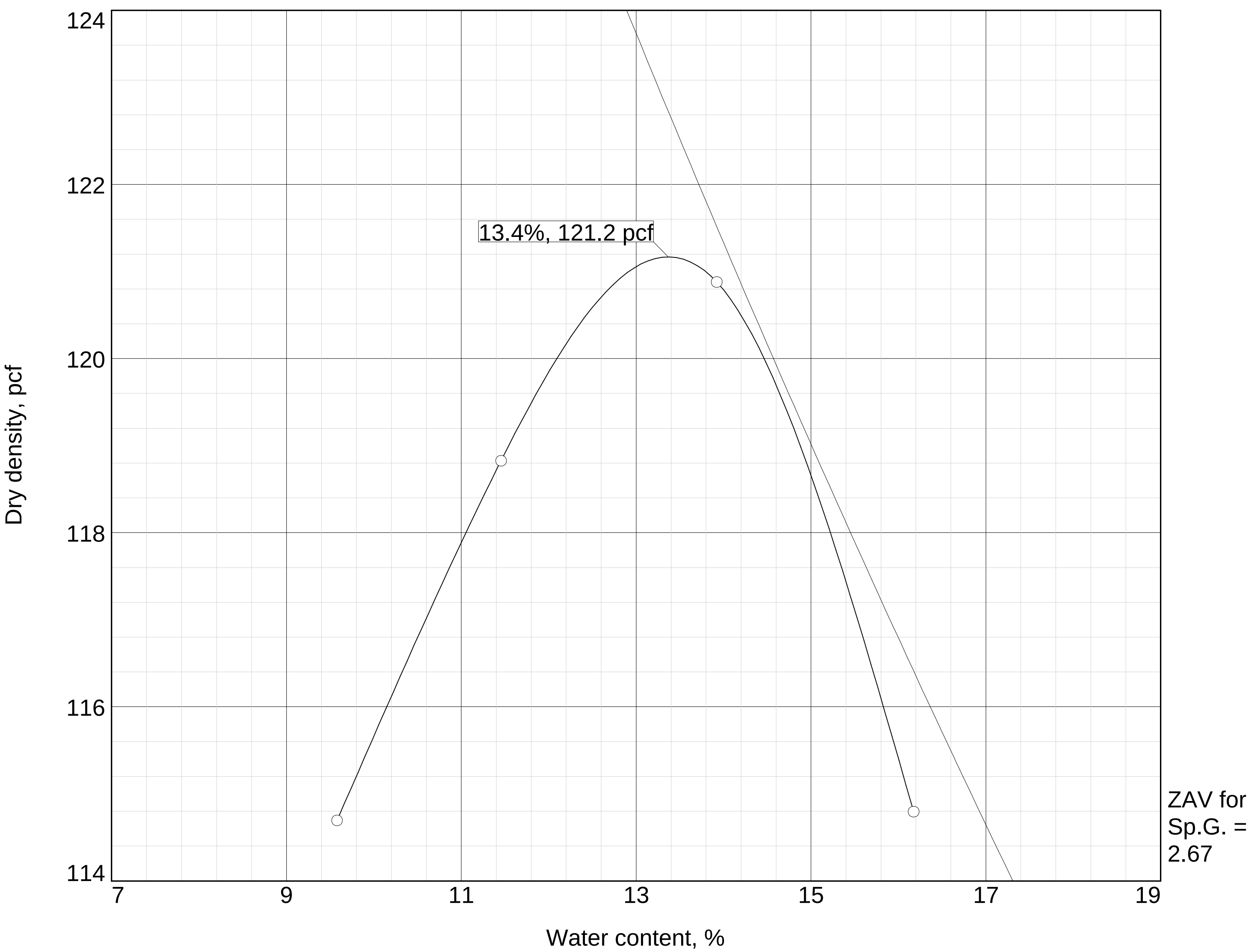
*

TABLE 1 Classification of Potential Expansion of Soils Using *EI*

Expansion Index, *EI*	Potential Expansion
0–20	Very Low
21–50	Low
51–90	Medium
91–130	High
>130	Very High

***ASTM D4829-11 pg.2, table 1**

COMPACTION TEST REPORT



Test specification: ASTM D 1557-12 Method A Modified, manual rammer, wet prep method

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
0.0-10.0'	SC			2.67			3	

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 121.2 pcf Optimum moisture = 13.4 %	CLAYEY SAND, dark yellowish brown

Project No. 3228.X **Client:** Stantec - Rocklin

Project: LWWTRF Expansion Phase 1&2

○ **Source of Sample:** B7 **Sample Number:** Bulks A&B

Blackburn Consulting

W. Sacramento, CA

Remarks:

Figure

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 02/02/2018
Date Submitted 01/30/2018

To: Rob Pickard
Blackburn Consulting (W.SAC)
2491 Boatman Ave
W. Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 3228.X LWWTRF Site ID : B7@0-10FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 76085-158684.

EVALUATION FOR SOIL CORROSION

Soil pH	7.31	
Minimum Resistivity	1.82 ohm-cm (x1000)	
Chloride	8.0 ppm	00.00080 %
Sulfate	23.9 ppm	00.00239 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

GEOTECHNICAL DESIGN REPORT

Lincoln Wastewater Treatment and Reclamation Facility

Phase 1 and Phase 2 Expansion Project
Maturation Pond Pump Station
Placer County, CA

APPENDIX C

Important Information About
This Geotechnical Engineering Report,
Geoprofessional Business Association, 2016

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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