



GEOTECHNICAL REPORT FOR THE EXPANSION OF POND 30

**Permanente Quarry, Lehigh Hanson Southwest
Cement**

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Table of Contents

- 1.0 INTRODUCTION..... 1
- 2.0 SITE SETTING AND GEOLOGIC CONDITIONS 2
 - 2.1 Topography 2
 - 2.2 Regional Geologic Setting..... 2
 - 2.3 Seismic Setting..... 3
 - 2.3.1 Structural and Tectonic Setting..... 3
 - 2.3.2 Seismicity 3
 - 2.4 Geologic Hazards..... 4
- 3.0 INVESTIGATION FOR POND 30 5
 - 3.1 Review of Aerial Photos 5
 - 3.2 Golder Field Exploration and Subsurface Conditions 5
 - 3.3 Laboratory Testing 6
 - 3.3.1 Test Methods 6
 - 3.3.2 Summary of Lab Test Results..... 6
- 4.0 SLOPE STABILITY ANALYSIS 8
 - 4.1 Methodology..... 8
 - 4.1.1 Static Analysis..... 8
 - 4.1.2 Seismic Analysis 8
 - 4.1.3 Critical Cross Section Analyzed..... 8
 - 4.2 Material Parameters..... 8
 - 4.3 Slope Stability under Static Conditions 9
 - 4.4 Slope Stability under Seismic Conditions 9
 - 4.4.1 Yield Coefficient for Seismic Slope Stability Analysis..... 9
 - 4.4.2 Seismically Induced Permanent Slope Displacement 10
- 5.0 CONCLUSIONS AND RECOMMENDATIONS..... 11
 - 5.1 Conclusions..... 11
 - 5.2 General Recommendations 11
- 6.0 USE OF THIS REPORT..... 13
- 7.0 CLOSING 14
- 8.0 REFERENCES..... 15

List of Tables

- Table 1 Summary of Material Parameters Used in the Stability Analyses
- Table 2 Summary of Static and Seismic Slope Stability Analyses Results

List of Figures

- Figure 1 Location Map





Figure 2	Conceptual Layout – Pond 30 Expansion
Figure 3	Geologic Map and Locations of Test Borings
Figure 4	Geologic Cross Section A-A'
Figure 5	Results of Static Slope Stability Analysis, Trial Surface Terminating at the Pond
Figure 6	Results of Static Slope Stability Analysis, Surficial Trial Surface within Existing Slope
Figure 7	Results of Pseudo-Static Slope Stability Analysis to Estimate Yield Acceleration

List of Appendices

Appendix A	Estimation of PGA and Acceleration Spectra for Maximum Credible Earthquake
Appendix B	Test Boring Logs
Appendix C	Laboratory Test Results
Appendix D	Slope Stability Analysis Results
Appendix E	Estimation of Seismically Induced Permanent Displacement



1.0 INTRODUCTION

Golder Associates Inc. (Golder) is submitting this geotechnical report for the proposed expansion of Pond 30 at Lehigh Hanson's Permanente Quarry located in Cupertino, California (Figure 1). This report summarizes the results of our field investigation, laboratory testing, and slope stability analyses.

The location of the proposed existing Pond 30 and the proposed grading for its expansion are shown in Figure 2. The pond is located along the southeast margin of the East Material Storage Area. Review of historical aerial photos show that the proposed pond expansion site is located on a level bench graded for construction of industrial buildings associated with the former Kaiser Aluminum Plant. The buildings were demolished in 2002, and the pad was used for an equipment lay-down and staging area, stockpile for aggregate materials, as well as for storm drainage management (i.e., existing pond 30 and associated inlet channel). The expanded pond has a surface area of approximately 1.5 acres with a storage depth of approximately 13 feet. The pond will be constructed entirely in excavation below existing ground surface. The approximate storage capacity of the pond is ~ 7 ½ acre-feet. The conceptual design of the pond includes a geomembrane liner to prevent infiltration.



2.0 SITE SETTING AND GEOLOGIC CONDITIONS

The following sections summarize the regional topographic, geologic, and seismic setting; and describe the specific geologic conditions in the area of Pond 30.

2.1 Topography

The Permanente Quarry and Cement Plant are situated in the foothills of the rugged, northwest-trending Santa Cruz Mountains segment of the California Coast Ranges. Topography in the area consists of moderately to steeply-sloped terrain with rounded ridges and drainages. Relief at the project site ranges from about 2000 feet along the higher ridge crests to less than 500 feet mean sea level (msl) along the eastern portions of Permanente Creek. Average natural slope angles are typically around 25°. The steepest natural slopes are on the order of 40° over smaller slope heights (100-200 feet) and generally correspond to limestone outcrops.

At the Pond 30 expansion area, the topography is a flat and level bench (el. ~560 feet amsl) created by excavation of a cutslope to the north, and placement of fill along the outboard edge of the pad. The pad was graded to accommodate construction of the former Kaiser Aluminum plant in the 1940's. The building were demolished in 2002. The pad is long and narrow (~2500 feet long by ~200 feet wide) and the alignment of the pad is parallel to the alignment of Permanente Creek.

2.2 Regional Geologic Setting

The majority of the subject property is underlain by complexly deformed and faulted rocks of the Franciscan Assemblage (Golder, 2011). The eastern portion of the site, is underlain in locations by Plio-Pleistocene rocks of the Santa Clara Formation. Overlying the bedrock are modern alluvial deposits associated with Permanente Creek (restricted to the eastern portion of the property), and relatively shallow surficial deposits comprised of soil and colluvium.

The Santa Clara Formation overlies the Franciscan Assemblage rocks in the central and eastern portion of the property including the EMSA area where it occurs as remnant patches of terrane overlying the Franciscan Assemblage and is locally in fault contact along the Sargent Berrocal fault. (Golder, 2011). The Santa Clara Formation is a continental fluvial and alluvial deposit that is composed of unconsolidated to slightly consolidated conglomerate, sandstone, siltstone, and claystone (Vanderhurst, 1981). Uplift of the Coast Ranges during this time resulted in increased erosion of the mountains and deposition of the Santa Clara Formation. The contact between the Franciscan rocks and Santa Clara Formation is considered to be unconformable, with the Santa Clara Formation deposited on an eroded Franciscan terrain (Rogers and Armstrong, 1973). Subsequent uplift of the nearby foothills along the Monte Vista fault, which lies along the margin of the valley floor to the east of the Quarry, has resulted in deformation of the Santa Clara Formation.



2.3 Seismic Setting

2.3.1 Structural and Tectonic Setting

The San Andreas Fault zone is located approximately 2 miles southwest of the quarry (Golder, 2011). The Sargent-Berrocal Fault Zone (SBFZ), part of the Santa Cruz Mountains front-range thrust fault system, parallels the San Andreas to the east and forms the eastern-most structural boundary to the Permanente Terrain.

Near the Permanente Site, the SBFZ consists of two northwest-trending, sub-parallel faults, namely the northeastern-most Monta Vista Fault Zone and the southwestern-most Berrocal Fault Zone (Sorg and McLaughlin, 1975). The combined fault zone is located in a complex contractional system of generally northeastward-vergent thrust and reverse faults that bound the northeastern side of the Santa Cruz Mountains (McLaughlin and others, 1997). This thrust system has been described as an eastward-propagating half-flower structure that roots toward the San Andreas fault zone.

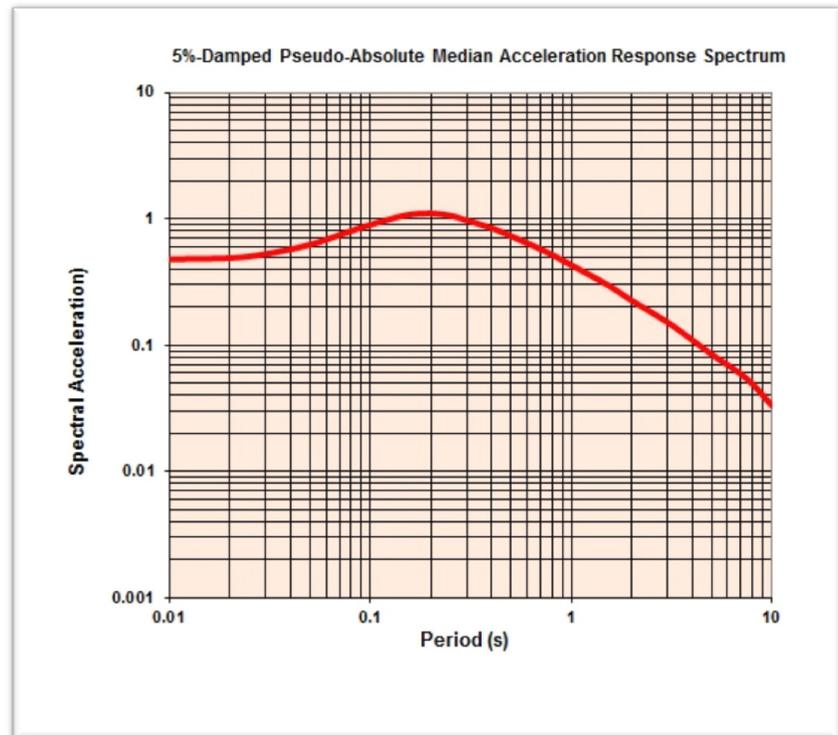
The Monte Vista strand of the fault zone is located approximately 800 feet to the northeast of the proposed Pond 30 expansion area. A strand of the Berrocal Fault Zone runs through the central portion of site and about 3100 feet west of the Pond 30 area (Sorg and McLaughlin, 1975). This fault forms a structural boundary between Franciscan basement rocks to the west, and Franciscan rocks overlain by Santa Clara formation rocks to the east. The Berrocal Fault Zone is not considered an active fault by the California Geologic Survey, it is classified as older than 1.6 million years (CGS, 2010). However, mapping of the Berrocal and Monte Vista faults following the 1989 Loma Prieta earthquake documented minor distributed coseismic contractional deformation in the Cupertino foothills to the northeast of the Permanente site (Hitchcock, et al, 1994).

2.3.2 Seismicity

The Permanente Site is located within the San Francisco Bay Area, which is a region characterized by relatively high seismicity. Golder evaluated potential seismic impacts for the project resulting from a maximum credible earthquake (MCE) on the San Andreas Fault. The MCE is defined as “the maximum earthquake that appears capable of occurring under the presently known tectonic framework.” The MCE would be a moment magnitude (M_w) 8 event along the San Andreas Fault, which is assumed to be slightly higher than the M_w 7.9 San Francisco earthquake of 1906.



Golder estimated the peak ground acceleration (PGA) and the acceleration spectra for the MCE using the Next Generation Attenuation (NGA) relationships developed by Abrahamson and Silva (2008), Boore and Atkinson (2008), Chiou and Youngs (2008), and Campbell and Bozorgnia (2008). The computed values from the four relationships were equally weighted (0.25 each) to estimate spectral accelerations as a function of magnitude, source-to-site distance, and fault geometry and for an average shear wave velocity in the upper 30 meters (100 feet) of the soil column (V_{s30}) equal to 760 meters per second (2,500 feet per second). The calculations are presented in Appendix A. Golder estimates that the design PGA is 0.48g for the site. The median acceleration spectrum is included here for reference.



2.4 Geologic Hazards

The geologic hazards evaluated for the proposed project include the potential for ground rupture, slope instability, liquefaction and lateral spreading, consolidation settlement, and potential flooding associated with the proposed project. The geologic hazards that could impact the proposed pond expansion are limited to ground shaking and slope instability.

The risk associated with ground rupture is considered negligible since the project site is not situated on a known Holocene fault. Liquefaction is not considered a hazard because of the presence of significant fines (silts and clays) in the soil underlying the proposed project site, and the relatively shallow depth to bedrock. Consolidation settlement is not considered a risk since the pond will be excavated and therefore the site will be unloaded and not subject to induced settlement. The site is located approximately 35 feet above the 100-year flood plain for Permanente Creek and therefore the risk associated with flooding is considered low. Tsunami and Seiche are not hazards due to the site location and elevation.

Slope instability, and the relative risk to the project, is discussed in more detail in Section 4.



3.0 INVESTIGATION FOR POND 30

3.1 Review of Aerial Photos

A review of various stereoscopic aerial photographs and Google Earth images dating back to 1948 was conducted to: (1) evaluate the past grading and development activities in the area, (2) prepare a geologic map of surficial deposits and bedrock, and (3) identify any obvious signs of ground distress related to slope instability.

The earliest set of aerial photographs (1948) show the Kaiser Aluminum buildings present on a graded pad. There are three buildings in the general area of the proposed Pond 30 expansion. There is relatively tall cutslope visible behind the northern two buildings. There is earthwork and grading activities associated with development of roads up to the buildings. The slope below the pad to the north, and the hillside above the buildings are covered with orchard and are native ground. Site conditions remain similar until late 2002 when the buildings were demolished and the land use changed to material stockpiles and equipment lay down. No evidence of slope instability was observed in the hillside terrain above or below the proposed pond expansion area.

3.2 Golder Field Exploration and Subsurface Conditions

On January 8-9, 2015, Golder performed a subsurface exploration consisting of four test borings at the proposed Pond 30 expansion site. The test borings are designated as B-1, B-2, B-3, B-4, and B-Pilot, the locations of which are shown in Figure 3.

The borings were advanced with a truck-mounted, CME 85 drill rig turning a continuous flight, 8-inch diameter, hollow-stem auger. During the drilling process, disturbed, but representative, soil samples were obtained at 5-foot intervals. The samples were obtained by using a 1.5-inch (ID) Standard Penetration Sampler and a California Modified Sampler driven by a 140-pound hammer falling freely for 30-inches. The number of hammer blows required to drive the sampler 18-inches were recorded in 6-inch intervals. The number of blows required to drive the sampler the final 12-inches is designated as the penetration resistance or "blow count." The blow counts as recorded in the field are presented on the boring logs.

The samples were retained in 1.5-inch and 2.5-inch diameter by 6-inch long brass tubes contained within the sampler. The samples were visually classified in the field by a geologist working under the direction of a California Professional Geologist (PG). The samples were retained in the tubes, placed in a sealed plastic bag inside a cooler, and transported to our office for sample review and selection of samples for laboratory testing.

All samples were classified in accordance to the Unified Soil Classification System. Pertinent information including depths, stratigraphy, soil engineering characteristics, and groundwater occurrence were



recorded. Stratigraphic contacts indicated on the logs represent approximate boundaries between soil types. The soil and groundwater conditions are those recorded for the dates indicated, and may not necessarily represent those of other times or locations.

Test borings B-1 and B-2 were extended to depths of 26.5 below ground surface (bgs) and B-3 and B-4 were drilled to 20 feet bgs. B-Pilot was drilled first to establish the general subsurface conditions and depth to bedrock. Representative bulk samples were obtained from this boring. Borings B-1 through B-4 were sampled using standard geotechnical procedures with Standard Penetration Test (SPT) drive samples and California Modified samples alternating every five feet.

The B-Pilot boring encountered highly weathered greenstone bedrock at approximately 30 feet below ground surface. Water was encountered at approximately 18 – 20 feet below ground surface although this water was possibly perched in this location. The upper four to five feet of the soil profile was described as Clayey Gravel (GW) and was interpreted as artificial fill. The remaining profile was described as Clayey Sand (SC) with gravel comprised of well graded, subrounded sand and gravel with iron oxidation. The material was compact and moist. This material is interpreted as colluvium overlying heavily weathered greenstone although it may be a mixture of artificial fill and colluvium as these earth materials are very similar in appearance and composition.

The remaining borings encountered the same general stratigraphic profile with minor variations in the interpreted depth of fill ranging up to about seven to eight feet in thickness. Bedrock was encountered at approximately 15 feet in Boring B-4. Groundwater was encountered at approximately 26 feet in Boring B-2. Logs of the test borings are included in Appendix B.

3.3 Laboratory Testing

3.3.1 Test Methods

Selected soil samples were transported to Cooper Testing Laboratories in Palo Alto, California for further classification and geotechnical testing. This testing included the following:

- Particle Size Analyses (ASTM D422)
- Atterberg Limits (ASTM D4318)
- Modified Proctor test (ASTM D1557)
- Direct shear test (ASTM D3080)

Laboratory test results are included in Appendix C.

3.3.2 Summary of Lab Test Results

The above laboratory tests were performed mainly on the colluvium samples. The colluvium classifies generally as *Lean Clayey SAND with Gravel*. The particle size analysis on a composite sample from



depth 3 to 25 feet from test boring B-1 shows 19.9% gravel, 48% sand, and 32.1% fines. The modified Proctor test on this sample shows a maximum dry density of 135.8 pounds per cubic foot (pcf) and an optimum moisture content of 8.1%.

The four Atterberg limit tests performed on four samples from various depths of the four borings show low plasticity with the liquid limit (LL) ranging from 28 to 37, the plastic limit (PL) ranging from 13 to 19, and the plasticity index (PI) ranging from 14 to 18.

The three consolidated drained direct shear strength tests show significantly large effective friction angles for the colluvium likely due to the presence of gravel. Because of the small size of the shear box relative to the size of the gravels, the measured shear strength appears to be affected by the gravel. Therefore, the direct shear test results are considered unconservative for the project.



4.0 SLOPE STABILITY ANALYSIS

4.1 Methodology

4.1.1 Static Analysis

The computer program SLOPE/W (2012 version) which uses two-dimensional, limit equilibrium method, was used to compute the factors of safety (FS) against potential slope failure at the site. This program allows both circular and noncircular sliding surfaces to be either defined manually or generated automatically to search for the lowest FS values. The Morgenstern-Price method was used to compute factors of safety.

Based on traditional geotechnical practice, a minimum FS of 1.5 is considered acceptable under static conditions.

4.1.2 Seismic Analysis

Golder performed a rigorous seismic slope stability analysis which included estimation of yield coefficient for the slope and estimating the seismically induced permanent displacement using a predictive model developed by Bray and Travasarou (2007). The yield coefficient is the seismic coefficient that results in pseudo-static FS of 1.

Based on the state-of-practice for seismic design of earthen slopes (Duncan and Wright, 2005), 3 feet is considered as the maximum allowable permanent of displacement during the MCE event. To be conservative, a seismically induces permanent displacement of 1 foot (12 inches) is assumed as the maximum allowable limit in this report.

4.1.3 Critical Cross Section Analyzed

Based on a review of the topography of the site and interpreted subsurface conditions at the Pod 30 site, Golder identified the most critical cross section (A-A') to analyze for stability of exiting slopes adjacent to the pond expansion. The location of this cross section is shown Figure 3. The geologic profile along cross section A-A' is shown in Figure 4.

4.2 Material Parameters

As shown in Figure 4, the subsurface profile along cross section A-A' consists of waste rock/artificial fill, fill/colluvium, and greenstone. The shear strength parameters for waste rock and greenstone were selected based on Golder's previous slope stability analyses for the East Material Storage Area in Appendix 11 of Golder (2011). For the fill/colluvium, Golder has assumed a conservative friction angle of 28 degrees, based on the type of materials encounters (i.e., *Lean Clayey SAND with Gravel*) and our engineering judgment. Table 1 below summarizes the parameters used in the slope stability analyses.

**Table 1: Summary of Material Parameters used in the Slope Stability Analyses**

Material	Unit Weight (pcf)	Shear Strength Parameters	
		Cohesion (psf)	Friction Angle (degrees)
Waste Rock/Artificial Fill ¹	125	0	35
Fill/Colluvium ²	120	0	28
Greenstone ¹	165	1,440	23

Notes:

¹ Based on Golder (2011)² Conservatively assumed value

4.3 Slope Stability under Static Conditions

The slope stability analysis was performed for two conditions using the SLOPE/W computer program. Because the pond will be lined with a geomembrane, the analyses assume that there would be groundwater recharge from the pond and would not raise the water table level or lead to seepage forces on the existing slopes located to the west of the pond.

The first slope stability condition analyzed is the potential for deep-seated slope failure surfaces that initiate near the toe of existing slope located to the west of the pond and terminating at the pond. The result of this analysis is shown in Figure 5. The analysis shows a FS of 1.82, which is greater than the minimum acceptable value of 1.5.

The second condition analyzed the potential for surficial sloughing of the relatively steep existing slopes located to the west of the pond. The result of this analysis is shown in Figure 6, which shows an acceptable FS of 1.54.

The results of SLOPE/W analyses are presented in Appendix D.

4.4 Slope Stability under Seismic Conditions

4.4.1 Yield Coefficient for Seismic Slope Stability Analysis

The yield coefficient (or yield accelerations) for each of the two conditions discussed in Section 4.3 were estimated from an iterative pseudo-static slope stability analysis using SLOPE/W by varying the input seismic coefficient. The result of the final iteration for deep-seated slope failure surfaces is shown in Figure 7, which shows a yield coefficient of 0.215g (where, g is the acceleration due to gravity). Similarly, the result of the final iteration for surficial sloughing is shown in Figure 8, which shows a yield coefficient of 0.17g.



The results of pseudo-static slope stability analyses are also presented in Appendix D.

4.4.2 Seismically Induced Permanent Slope Displacement

The likely magnitude of permanent displacement was estimated using the Bray and Travasarou (2007) method. The permanent displacement calculations are presented in Appendix E. The calculations for deep-seated failure surface estimated a permanent displacement of 9.3 inches during the magnitude 8 MCE. Similarly, for surficial sloughing, the estimated permanent displacement is 4.3 inches. Both of these values are less than the allowable limit of 12 inches discussed in Section 4.1.2.

Table 2 below presents a summary of the static and seismic slope stability analyses.

Table 2: Summary of Static and Seismic Slope Stability Analyses Results

Condition Analyzed	Static FS	Yield Coefficient	Estimated Permanent Displacement (inches)
Deep-Seated Failure	1.82	0.215g	9.3
Surficial Sloughing	1.54	0.170g	4.3



5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

It is Golder's opinion that the site is suitable for development of the proposed Pond 30 expansion project.

The results of the static and seismic slope stability analyses discussed in this report show that the expansion of Pond 30 will not adversely impact stability of the existing slopes either above or below the ponds. For the pond expansion itself, the static slope stability analysis shows a factor of safety of greater than the acceptable minimum values of 1.5 for a slip circle intercepting the pond liner. Similarly, the seismic slope stability analyses estimated seismically induced permanent displacement of less than the maximum allowable limit of 12 inches.

Based on the encountered soil types and relatively shallow depth to bedrock, there is a low risk of potential liquefaction. The site is situated approximately 35 feet above the 100-year flood plain for Permanente Creek so there is a low risk of flooding. Although currently deeper, there is the potential that groundwater could rise to the proposed depth of the pond excavation (~20 feet) and should be accounted for in the final design for the project with appropriate recommendations for underdrains.

5.2 General Recommendations

The following provide general recommendations based on our investigation and the conceptual plan for the project. Golder will provide more detailed geotechnical recommendations and specifications in conjunction with preparation of final designs and construction plans for the project as necessary.

- **Construction Observation:** All earthworks should be observed and tested by a qualified geotechnical engineering company. Construction observation and testing services may include, but not be limited to, foundation subgrade verification, and verification that the placement and compaction of engineered fill complies with recommendations and specifications.
- **Site Preparation:** Prior to excavation and placement of artificial fill, the project area should be stripped to remove the existing vegetation. Golder anticipates that the depth of stripping will be 4- to 6-inches or less. Stripped soils may be stockpiled for re-use as vegetative layer soils.
- **Earthworks Grading:** Site grading is anticipated to primarily consist of the excavation of site soils to create the pond expansion, with more limited placement of engineered fills to achieve desired site grades. The subgrade should be prepared to achieve a firm and unyielding condition. All exposed soil surfaces that will receive fill or pavements should be moisture conditioned as needed and compacted prior to fill placement or surfacing application. Scarification of near-surface soils may be necessary for moisture-conditioning, but in no case should scarification extend deeper than 12 inches. Soil should be compacted to the densities provided in the specifications for the project.

Care should be taken to avoid disturbing subgrade soils and foundation soils that will remain in place. Areas which become softened or disturbed during construction should be moisture conditioned and recompacted or removed and replaced with properly placed



and compacted structural fill. Prior to fill placement, fill subgrades should be proof-rolled with a loaded water truck or dump truck to identify yielding conditions. Yielding soils should be moisture-conditioned and recompacted, or excavated and replaced with structural fill.

- **Fill Materials and Placement:** Engineered fill derived from on-site sources may be used to construct the proposed site earthworks. Engineered Fill materials should consist of well graded soils with a maximum particle size of 4 inches and free of organic material, trash or other deleterious materials. Based on the existing laboratory data, the local borrow soils will generally meet the requirements for Engineered Fill.

Engineered Fill materials should be placed in a maximum 6-inch loose thick lift thickness and be compacted to a maximum relative compaction of at least 90 percent and at a moisture content of between -1 and +4 percent of the optimum moisture content in accordance with ASTM D 1557 (Modified Proctor moisture-density relationship).

If density tests taken in the Engineered Fill indicate that compaction is not being achieved due to high or low moisture content, then the fill should be scarified, moisture-conditioned, and recompacted. If the required densities cannot be met then the material should be over-excavated and replaced with a suitable material, or if the soils are excessively wet, a soil admixture used to dry the soil.

Earthwork construction during wet weather may significantly increase costs associated with off-site disposal of unsuitable excavated soils, increased control of water, and increased problems with subgrade disturbance and need for soil admixtures, geotextiles, rock working mats, or other stabilization measures to address unsuitable soil conditions. It is therefore recommended that the project earthwork be completed during the dry season (i.e., prior to November 1 annual onset of the rainy season).

- **Temporary and Permanent Slopes:** Golder recommends that permanent Engineered Fill slopes be 2.5H:1V or flatter. Proposed permanent cutslopes should be 2H:1V or flatter.

Permanently exposed slopes should be seeded with an appropriate species of vegetation or covered with an appropriate armoring to reduce erosion and improve stability of the surficial layer of soil. Slopes may experience erosion or sloughing if not well vegetated or covered. In the event that the cuts and fills exceed 20 feet in height, Golder should be contacted so that we can review and revise our recommendations if necessary.

The inclination of temporary slopes is dependent on several variables, including the height of the cut, the soil type and density, the presence of groundwater seepage, construction timing, weather, and surcharge loads from adjacent structures, roads and equipment. In no case should excavation slopes be greater than the limits specified in local, state (Cal-OSHA), and federal (OSHA) safety regulations. Safe temporary slopes are the responsibility of the contractor and should comply with all applicable OSHA and state standards.

- **Pond Lining:** To minimize the potential for damage to the geomembrane liner and consequent leakage, it is recommended that a 0.5-foot-thick bedding layer consisting of soil having particle size no greater than 4.76 millimeters (0.19 inches) be used below the geomembrane. The geomembrane should be tested for leaks and repaired if damage is detected. Underdrains should be considered for the final design as groundwater could be encountered at 20 feet bgs or less.



6.0 USE OF THIS REPORT

This report has been prepared for the exclusive use of Lehigh Hanson Southwest Cement Company for specific application to the proposed expansion of Pond 30. Golder is not responsible for any unauthorized use of this report.

The findings, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice that exists within the area at the time of the work. No other warranty, expressed or implied, is made.

The analyses and recommendations contained in this report are based on data obtained from the results of previous subsurface explorations by others as well as the site reconnaissance and explorations conducted by Golder. The methods used generally indicate subsurface conditions at the time and locations explored and sampled. In addition, groundwater conditions can vary with time.



7.0 CLOSING

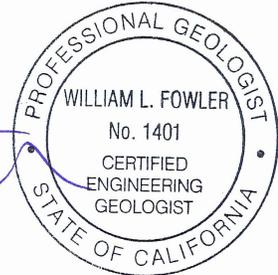
We appreciate the opportunity to support the Lehigh Hanson Southwest Cement Company on this project. Please call Bill Fowler at (408) 220-9239 if you have any questions, or require clarification of our findings and recommendations.

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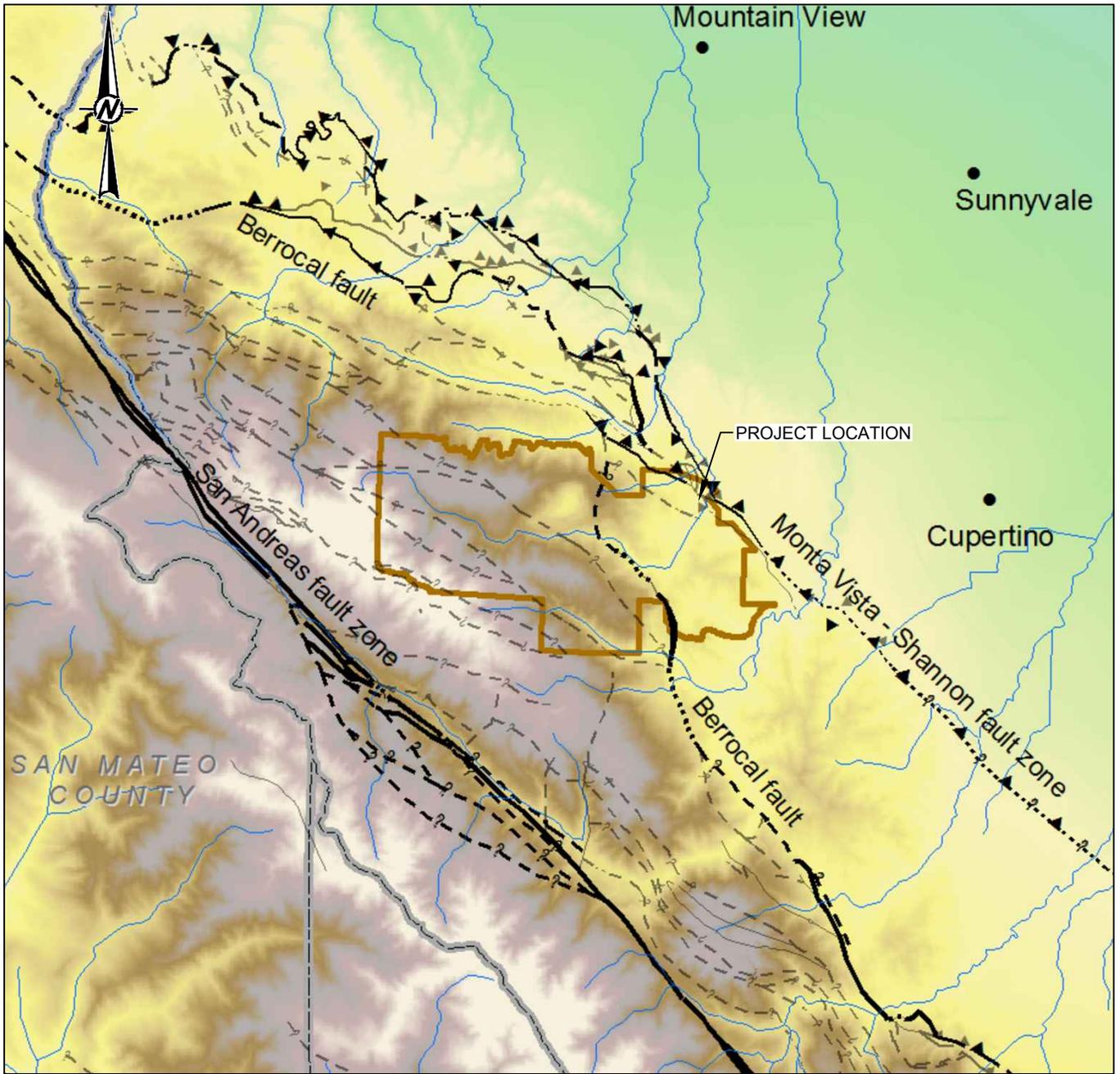
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NOTES

1. TOPOGRAPHIC MAP OBTAINED FROM THE USGS NATIONAL MAP (www.nationalmap.org).
2. IMAGE NOT TO SCALE.

LEGEND

- ?---?--- INFERRED FAULT
- ▲▲▲▲▲▲ THRUST (TEETH IN DIP DIRECTION)
- CONCEALED FAULT
- SITE BOUNDARY

CLIENT
**LEHIGH HANSON SOUTHWEST CEMENT
 PERMANENTE PLANT
 SANTA CLARA COUNTY, CALIFORNIA**

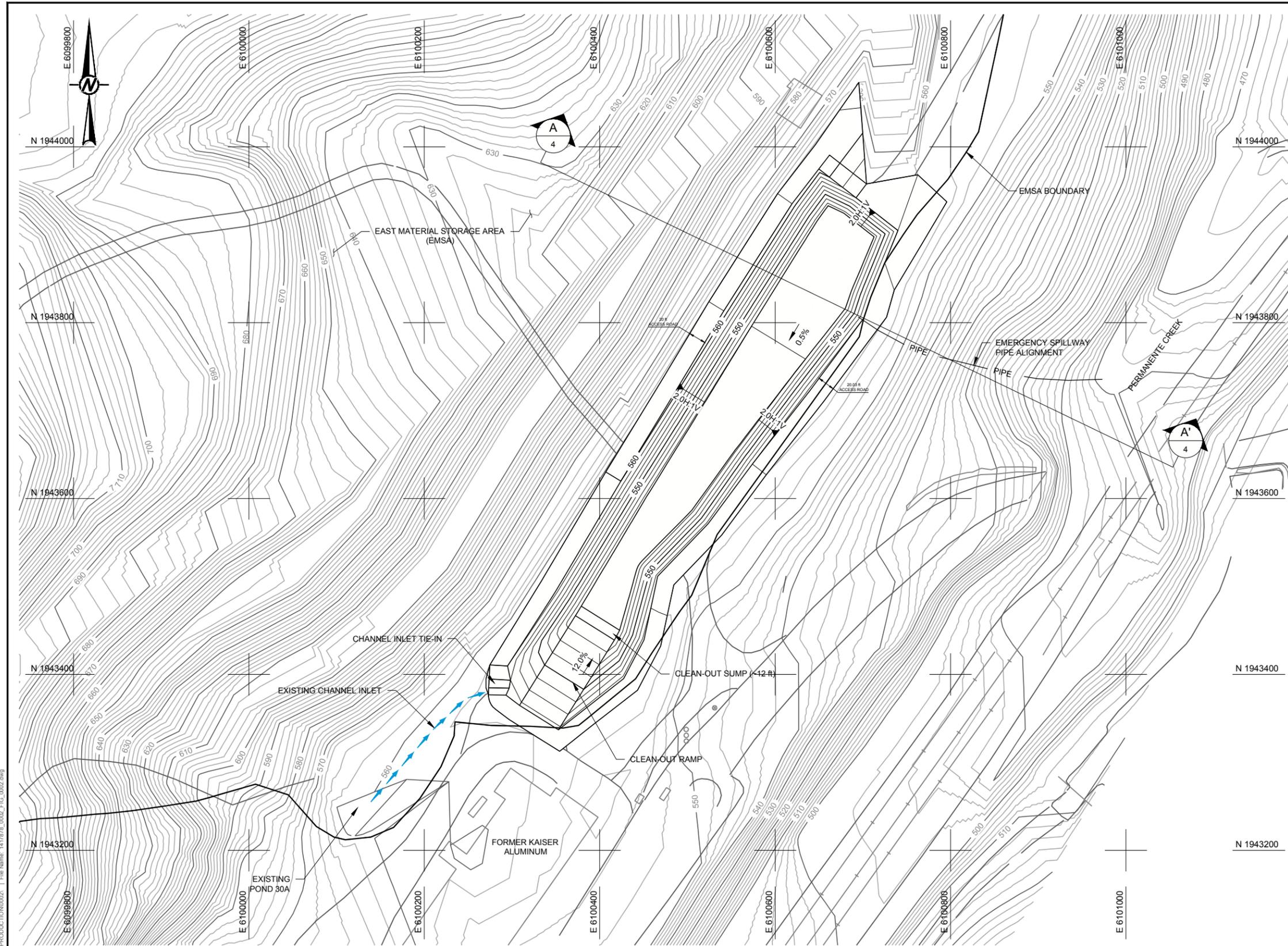
PROJECT
**GEOTECHNICAL REPORT
 FOR THE EXPANSION OF POND 30**

CONSULTANT	YYYY-MM-DD	2015-02-20
	PREPARED	JHR
	DESIGN	CDJ
	REVIEW	NK
	APPROVED	WLF

TITLE
LOCATION MAP

PROJECT No.	CONTROL	Rev.	FIGURE
1417878	0002	0	1





- GENERAL LEGEND**
- EXISTING GROUND CONTOUR (ft -MSL)
 - EXISTING ROADS
 - EXISTING FENCELINE
 - EXISTING POWER POLE OR POST
 - DESIGN CONTOURS (ft -MSL)
 - GRADE BREAK
 - EMSA BOUNDARY
 - SLOPE INDICATOR
 - EMERGENCY SPILLWAY PIPE ALIGNMENT
 - 3H:1V or $\frac{3H}{1V}$ 3 HORIZONTAL TO 1 VERTICAL SLOPE
 - GRADE INDICATOR
 - CHANNEL FLOW
 - CROSS-SECTION CALLOUT
SECTION ID
DRAWING SHEET LOCATION



Path: \\nasas\cadd\2014\Projects\1417878\Lehigh_Pond_30\PRODUCT\FIG_0002.dwg | File Name: 1417878_0002_FIG_0002.dwg

CLIENT
LEHIGH HANSON SOUTHWEST CEMENT
PERMANENTE PLANT
SANTA CLARA COUNTY, CALIFORNIA

CONSULTANT	YYYY-MM-DD	2015-02-20
	PREPARED	JHR
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	APPROVED	WLF

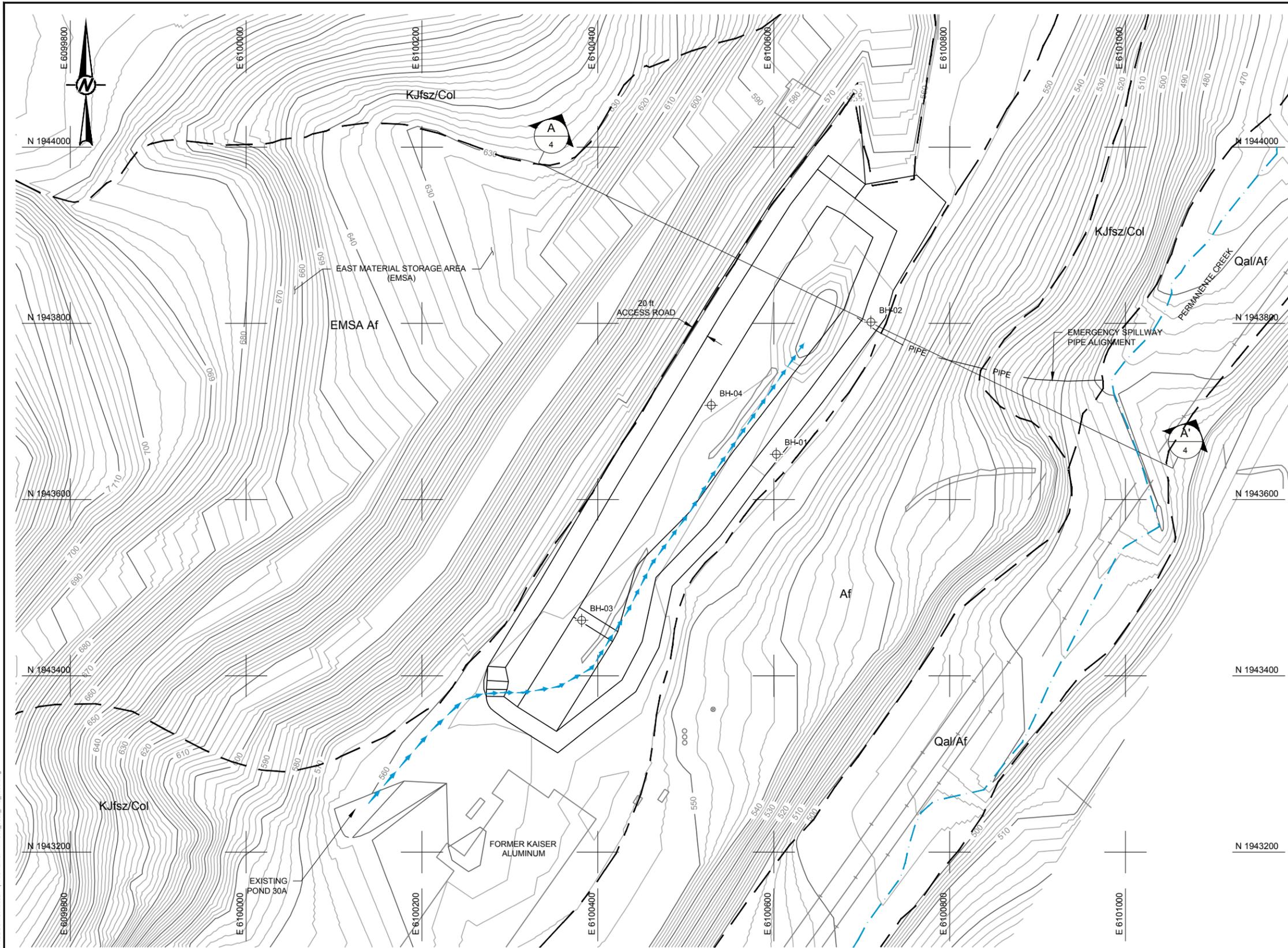


PROJECT
GEOTECHNICAL REPORT
FOR THE EXPANSION OF POND 30

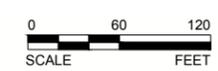
TITLE
CONCEPTUAL LAYOUT - POND 30 EXPANSION

PROJECT No.	CONTROL	Rev.	FIGURE
1417878	0002	0	2

1in IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM ANSI B



- GENERAL LEGEND**
- EXISTING GROUND CONTOUR (ft -MSL)
 - EXISTING ROADS
 - EXISTING FENCELINE
 - GRADE BREAK
 - EXISTING POWER POLE OR POST
 - BOREHOLE LOCATION
 - GEOLOGIC CONTACT, APPROXIMATE
 - GEOLOGIC CONTACT, CONCEALED
 - PERMANENTE CREEK
 - PIPE
 - EMERGENCY SPILLWAY PIPE ALIGNMENT
 - CHANNEL FLOW
 - CROSS-SECTION CALLOUT SECTION ID DRAWING SHEET LOCATION
 - Qal/Af** MIXTURE OF ALLUVIUM AND ARTIFICIAL FILL ALONG ENGINEERED PERMANENTE CREEK ALIGNMENT
 - Af** ARTIFICIAL FILL
 - KJfsz/Col** MELANGE OF FRANCISCAN COMPLEX ROCKS METABASALTS (GREENSTONE), SANDSTONE (GREYWACKE) WITH VARYING THICKNESSES OF COLLUVIUM
 - EMSA Af** OVERBURDEN ROCK FILL MATERIALS



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CLIENT
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PERMANENTE PLANT
SANTA CLARA COUNTY, CALIFORNIA

CONSULTANT	YYYY-MM-DD	2015-02-20
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	APPROVED	WLF

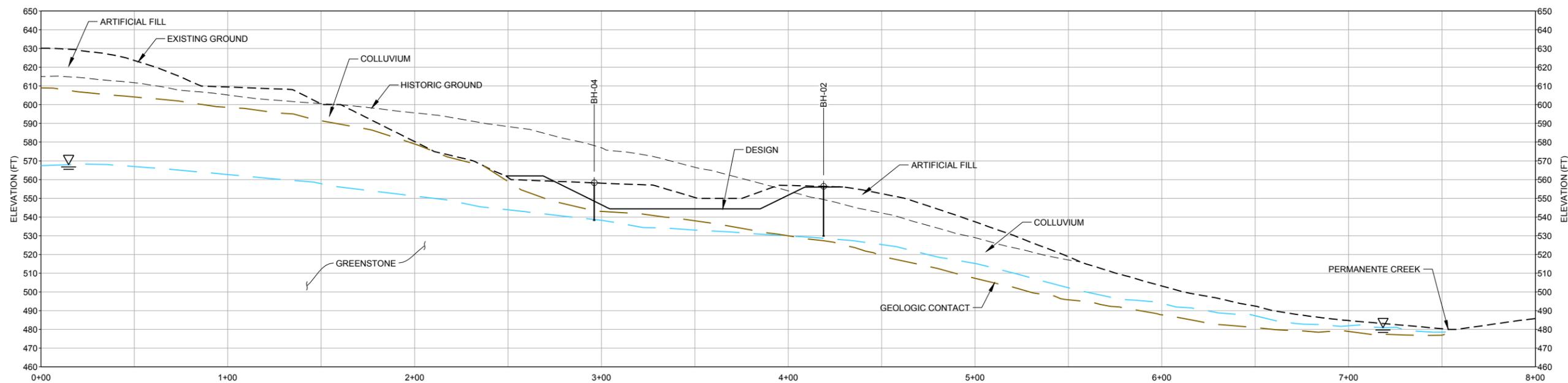


PROJECT
GEOTECHNICAL REPORT
FOR THE EXPANSION OF POND 30

TITLE
GEOLOGIC MAP AND LOCATION OF TEST BORINGS

PROJECT No.	CONTROL	Rev.	FIGURE
1417878	0002	0	3

1in IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM ANSI B



- LEGEND**
- EXISTING GROUND
 - DESIGN SURFACE
 - HISTORIC GROUND, APPROXIMATE
 - WATER LEVEL, APPROXIMATE
 - GEOLOGIC CONTACT, APPROXIMATE



CLIENT
**LEHIGH HANSON SOUTHWEST CEMENT
 PERMANENTE PLANT
 SANTA CLARA COUNTY, CALIFORNIA**

CONSULTANT	YYYY-MM-DD	2015-02-20
	PREPARED	JHR
	DESIGN	CDJ
	REVIEW	NK
	APPROVED	WLF



PROJECT
**GEOTECHNICAL REPORT
 FOR THE EXPANSION OF POND 30**

TITLE
GEOLOGIC CROSS-SECTION A-A'

PROJECT No.	CONTROL	Rev.	FIGURE
1417878	0002	0	4

POND 30 UPGRADE FEASIBILITY DRAWINGS

LEHIGH SOUTHWEST CEMENT CO. PERMANENTE PLANT SANTA CLARA COUNTY, CALIFORNIA FEBRUARY 2015



NOTE: TOPOGRAPHIC MAP OBTAINED FROM THE USGS NATIONAL MAP (www.nationalmap.gov).

SITE VICINITY MAP
NOT TO SCALE

INDEX OF SHEETS

No.	DRAWING TITLE	
		GENERAL
G-0001	TITLE SHEET	
G-0002	GENERAL ARRANGEMENT	
		CIVIL
C-0001	PLAN VIEW	
C-0002	SECTIONS	

GENERAL NOTES

- CONTRACTOR IS RESPONSIBLE FOR APPROPRIATE ON-CALL UTILITY LOCATE PROCEDURES PRIOR TO EXCAVATION. IF A SUBSURFACE UTILITY IS ENCOUNTERED IN THE EXCAVATION, WORK IN THAT AREA WILL BE STOPPED AND LSCC'S PROJECT MANAGER WILL BE NOTIFIED IMMEDIATELY. WORK IN THAT AREA WILL NOT RESUME UNTIL DIRECTED BY LEHIGH'S PROJECT MANAGER.
- EXCAVATION SLOPE CALLOUTS ILLUSTRATED ON DRAWINGS ARE CONSIDERED TYPICAL.
- CONTRACTOR IS RESPONSIBLE FOR SLOPING EXCAVATIONS TO MAINTAIN SAFE WORKING CONDITIONS IN ACCORDANCE WITH APPLICABLE STANDARDS.
- TEMPORARY EROSION CONTROL SYSTEMS ARE TO BE PLACED AS FIELD DETERMINED BY CONTRACTOR AND CONSTRUCTION MANAGER TO PROTECT EROSION PRONE AREAS.
- THE MAXIMUM GROSS WEIGHT OF EQUIPMENT DESIGNED AT THE POND 30 UPGRADE FACILITY IS 80,000 POUNDS AND THE MAXIMUM WIDTH IS BETWEEN 12 AND 15 FEET DEPENDING ON THE MSHA REQUIRED SAFETY BERM HEIGHT.

GENERAL REFERENCE

- TOPOGRAPHY USED IN THE GENERAL SITE LAYOUT REPRESENTS AERIAL SURVEYS BETWEEN 2009 TO JUNE 2013 PROVIDED BY LEHIGH SOUTHWEST CEMENT CO.
- EXISTING GROUND TOPOGRAPHY USED IN DESIGN OF POND 30 FROM A JANUARY 7, 2015 SURVEY PROVIDED BY LEHIGH SOUTHWEST CEMENT CO.
- COORDINATE SYSTEM IS IN CALIFORNIA STATE PLANE ZONE 3 NAD 83 (2014).

NOT FOR CONSTRUCTION

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Rev.	YYYY-MM-DD	DESCRIPTION	PREPARED	DESIGN	REVIEW	APPROVED
0	2015-02-27	ISSUED FOR FINAL REPORT	NIL	CDJ	CDJ	WLF
B	2015-01-13	ISSUED FOR CLIENT REVIEW	JHR	CDJ	CDJ	WLF
A	2015-01-12	ISSUED FOR INTERNAL REVIEW	JHR	CDJ	CDJ	WLF

SEAL

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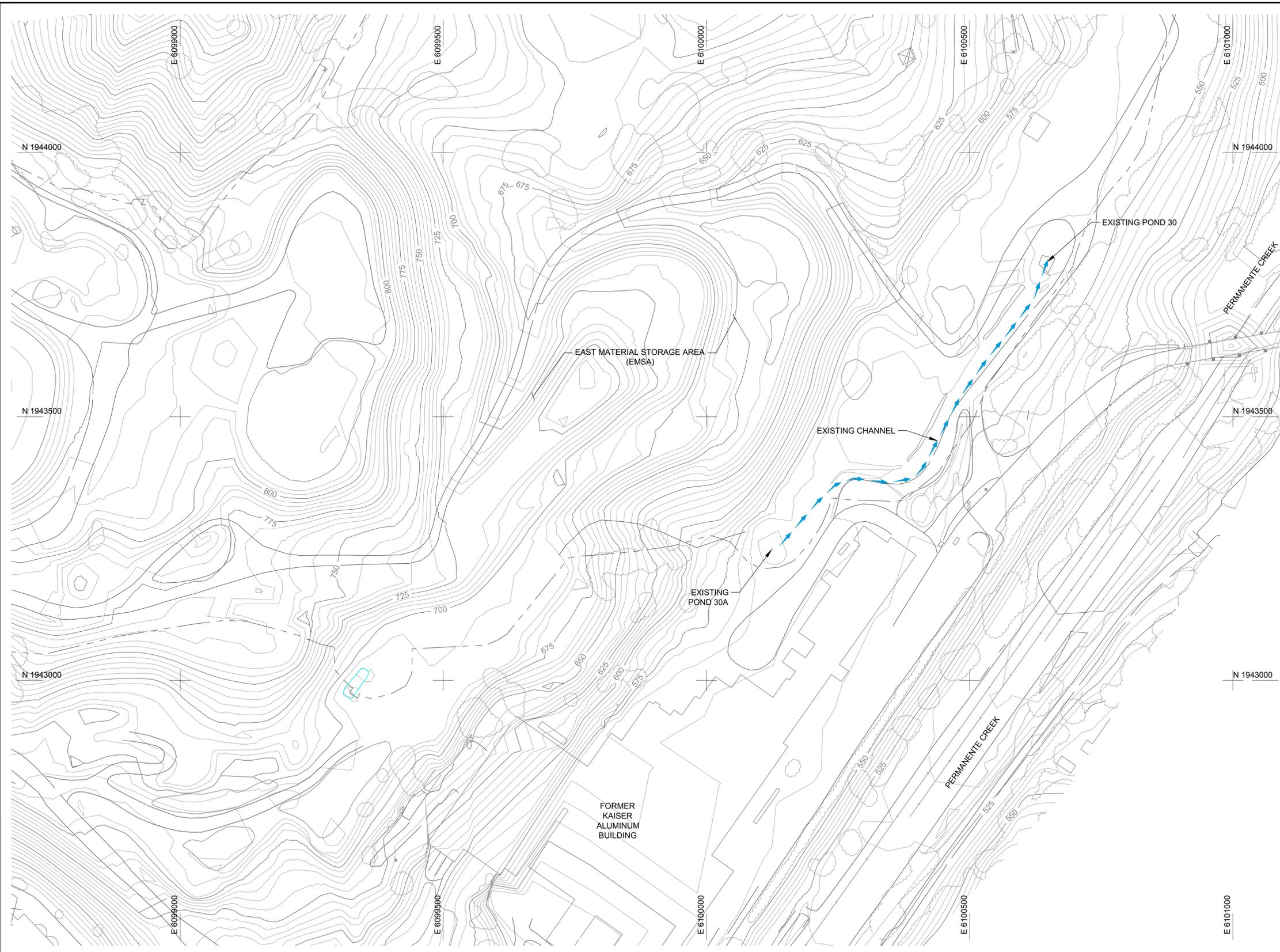


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PROJECT
POND 30 UPGRADE
FEASIBILITY DRAWINGS

TITLE
TITLE SHEET

PROJECT No. 1417878 CONTROL 0001 Rev. 0 1 of 4 FIGURE G-0001



- GENERAL LEGEND**
- EXISTING GROUND CONTOUR (ft.-MSL)
 - EXISTING ROADS
 - EXISTING FENCELINE
 - EXISTING POWER POLE OR POST
 - DESIGN CONTOURS (ft.-MSL)
 - GRADE BREAK
 - SLOPE INDICATOR
 - 3 HORIZONTAL TO 1 VERTICAL SLOPE
 - GRADE INDICATOR
 - CHANNEL FLOW
 - CROSS-SECTION CALLOUT
SECTION ID
DRAWING SHEET LOCATION

NOT FOR CONSTRUCTION



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0	2015-02-27	ISSUED FOR FINAL REPORT	NIL	CDJ	CDJ	WLF
B	2015-01-13	ISSUED FOR CLIENT REVIEW	JHR	CDJ	CDJ	WLF
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SEAL

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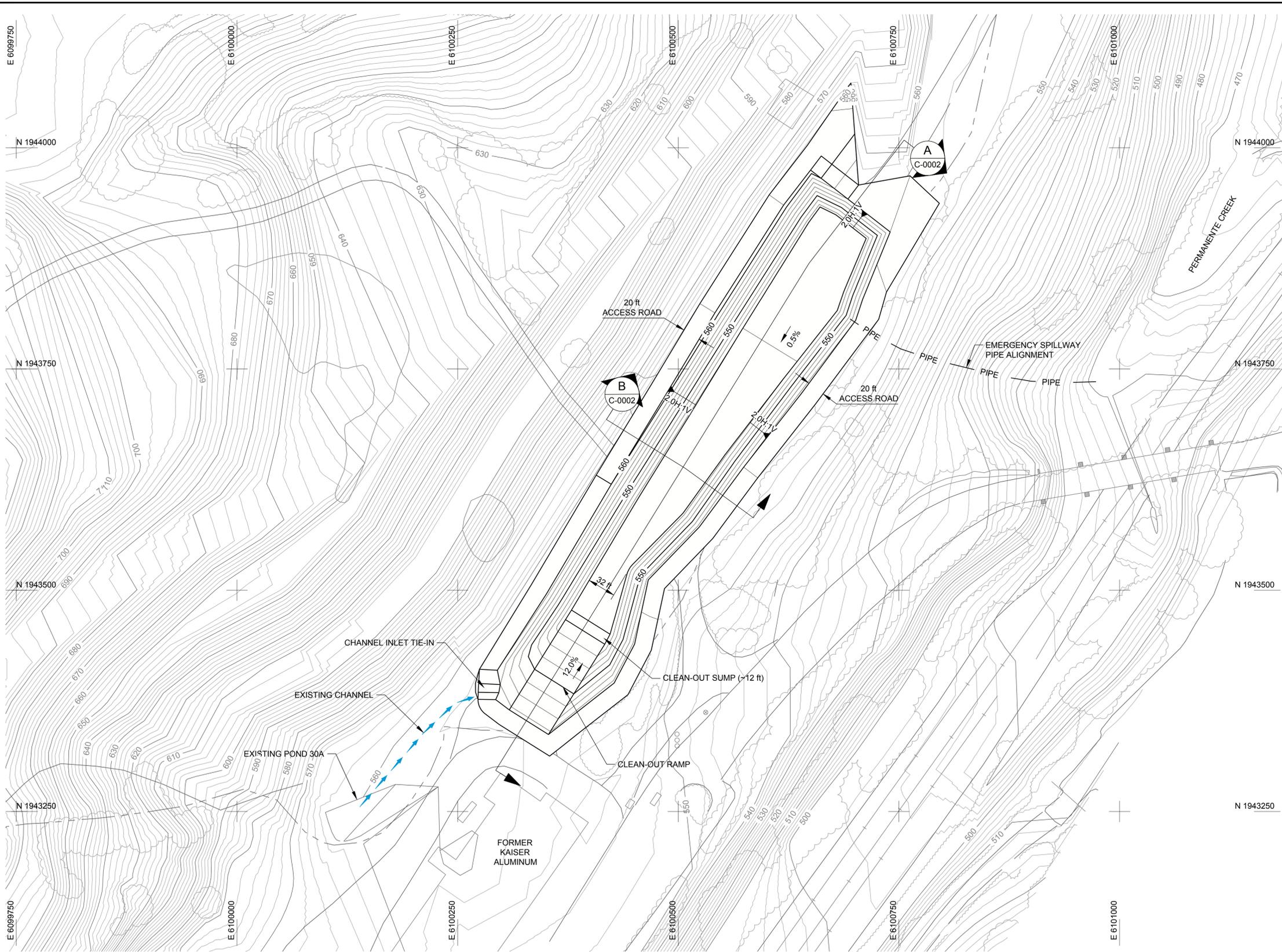


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PROJECT
POND 30 UPGRADE
 FEASIBILITY DRAWINGS

TITLE
GENERAL ARRANGEMENT

1 in IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI D



- GENERAL LEGEND**
- EXISTING GROUND CONTOUR (ft -MSL)
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 - 3 HORIZONTAL TO 1 VERTICAL SLOPE
 - GRADE INDICATOR
 - CHANNEL FLOW
 - CROSS-SECTION CALLOUT SECTION ID DRAWING SHEET LOCATION

NOT FOR CONSTRUCTION



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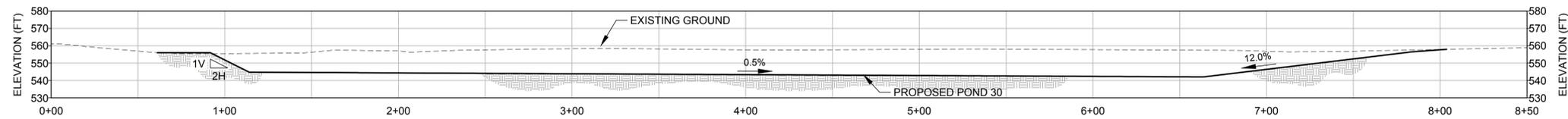
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FEASIBILITY DRAWINGS

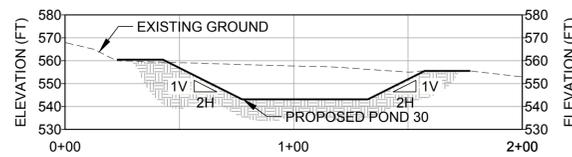
TITLE
PLAN VIEW

PROJECT No. 1417878 CONTROL 0001 Rev. 0 3 of 4 FIGURE C-0001

1 in IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM AASD



SCALE A **A** CROSS-SECTION A
C-0002



SCALE A **B** CROSS-SECTION B
C-0002

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PROJECT
POND 30 UPGRADE
FEASIBILITY DRAWINGS

TITLE
CROSS-SECTIONS

Rev.	YYYY-MM-DD	DESCRIPTION	PREPARED	DESIGN	REVIEW	APPROVED
0	2015-02-27	ISSUED FOR FINAL REPORT	NIL	CDJ	CDJ	WLF
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A	2015-01-12	ISSUED FOR INTERNAL REVIEW	JHR	CDJ	CDJ	WLF

PROJECT No. 1417878 CONTROL 0001 Rev. 0 4 of 4 FIGURE C-0002

Pond 30 (EMSA) catchment parameters			
Area (ft ²)	Area (acres)	Curve Number	S
4,454,059	102.3	70	4.29

Month	Average Precip ¹ (in)	Unit Runoff, Q (in)	Incremental Runoff Volume (ac-ft)	Cumulative Runoff Volume (ac-ft)	Pond Stage (ft)
October	0.89	0.00	0.00	0.00	542.45
November	2.92	0.67	5.71	5.71	549.46
December	4.25	1.50	12.77	18.49	Overflow
January	3.37	0.93	7.91	26.40	Overflow
February	5.59	2.48	21.16	47.57	Overflow
March	2.90	0.66	5.62	53.18	Overflow
April	1.68	0.13	1.13	54.31	Overflow
May	0.46	0.04	0.35	54.66	Overflow
June	0.08	0.17	1.47	56.13	Overflow
July	0.00	0.00	0.00	56.13	Overflow
August	0.01	0.21	1.78	57.91	Overflow
September	0.06	0.18	1.55	59.46	Overflow

Notes:

1 Average annual precipitation data from Los Altos Hills weather station (1999-2006 & 2009).