West Valley Feeder No. 1 Stage 3 Improvements Project

Proposed
Initial Study-Mitigated Negative Declaration

Appendices D through G

Metropolitan Report No. 1582

June 2024

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The Metropolitan Water District of Southern California
700 North Alameda Street
Los Angeles, California 90012

Report No. 1582

June 2024

APPENDICES

Appendix

APPENDIX D

ARCHAEOLOGICAL INVENTORY REPORT

ARCHAEOLOGICAL INVENTORY
Metropolitan Water District (MWD) West Valley Feeder No 1 (WVF1) Stage 3 Improvements Project

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Abstract

The Metropolitan Water District of Southern California (MWD) has requested an archaeological record search and inventory for the proposed construction of an approximately 500-foot access road including a vehicle turn-around area and various modifications to existing facilities including valve relocation, equipment replacement, and reconstruction of valve structures. The archival research indicated that the project area is sensitive for archaeological resources with multiple sites in the immediate area. The foot reconnaissance was conducted and found ground visibility to be poor and could not determine if archaeological resources were present in the access road alignment. No archaeological resources were observed in the other impact areas. The proximity of recorded archaeological resources coupled with poor ground visibility warrants a recommendation for monitoring by an archaeological and Native American monitor.

Should potentially important cultural deposits be encountered during ground disturbing activities, work should be temporarily diverted from the vicinity of the discovery until the archaeologist and Native American can identify and evaluate the importance of the find, conduct any appropriate assessment, and implement measures to mitigate impacts on significant resources.

USGS Quadrangles: Oat Mountain and Santa Susana
Acreage: Various acres
Cultural Resources: None observed
Type of Investigation: Archaeological Record Search and Inventory

Cover Picture: Aerial view of subject area.

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This report is not for public distribution

INTRODUCTION

Greenwood and Associates has conducted an archaeological record search and field inventory for the proposed Project for Metropolitan Water District (MWD) West Valley Feeder No. 1 (WVF1) Stage 3 Improvements Project in the community of Chatsworth in Los Angeles (Figure 1).

The study was prepared in order to identify any archaeological resources within the proposed impact areas. The investigation provides the necessary documentation to satisfy its obligations relative to CEQA requirements. The effort included a review of available archaeological site archives, historical maps, documents describing the proposed project area, and a survey of previously identified archaeological sites. This report describes the results of the background research, methods and results of the field investigation, and conclusions regarding the probability of impact to cultural resources due to project-related activities.

The Project involves modification of the MWD WVF1 structures, which is-located northwest of Chatsworth Park South, in the City of Los Angeles. Proposed project actions include construction of an approximately 14-foot wide by 500-foot long access road including a vehicle turn-around area and various modifications to existing structures including valve relocation, equipment replacement, and reconstruction of valve structures. Additionally, the project proposes the installation of new manholes at existing structures, a concrete vault, and retaining walls along the WVF1. Project impacts would include both temporary impact areas associated with construction access, staging, and laydown areas as well as permanent impacts associated with the proposed access road. Except for those areas where impacts would be confined to existing structures and the surrounding, paved areas, all other impact areas occurring would be subject to some degree of earth disturbance (Figure 2).

CURRENT SETTING

The project area is on and within the east facing hills of the community of Chatsworth within the city of Los Angeles. The hills are covered in chaparral, sandstone cliffs, boulders, paved roads, lightly graded roads, and trails. MWD facilities including structures, pipelines, and other facilities are dispersed throughout the area. Las Virgenes and Calleguas Water Districts have pump stations and pipelines in Chatsworth Park. Lower portions of the park recently underwent extensive lead soil remediation.

The West Valley Feeder No. 1 was constructed in 1962 and has an inside diameter of 54 inches. Specific installation methods and exact excavation depths vary from pipeline to pipeline; however, the excavation methods and typical disturbance areas can be described. Generally pipelines have 5 to 10 feet of cover to the top of the pipe, although in some areas it may be substantially more due to topography or to avoid existing facilities. In undeveloped areas, such as the project area, trenching was generally open cut excavation with 1:1 side slopes. Shoring is used in developed areas and along public streets. In the areas where open cut excavation is employed, the trench depths are generally between 15 and 20 foot deep and 30 to 50 foot wide at the existing ground surface, depending on topography.

BACKGROUND

Ethnography

This section summarizes the regional and cultural history of the project area. The discussion has been limited to that Native American group described as occupying the project area at the time of European contact and the historically documented activities following that contact. Chatsworth was inhabited by the Tongva-Fernandeño, Chumash-Venturaño, and Tataviam-Fernandeño Native American tribes.

Prehistory

The archaeological record indicates that sedentary populations occupied the coastal and inland regions of California more than 13,000 years ago. Early periods were characterized by the processing of hard seeds with the mano and milling stone and the use of the atlatl (dart thrower) to bring down large game, e.g., deer. Villages in eastern Ventura area were typically around permanent water sources that allowed exploitation of a variety of different habitats for food. In the later periods, prior to the arrival of Europeans, the bow and arrow was in use, trade and social networks evolved, and the mortar and pestle were used to process acorns in areas where they were available.

At the time of European contact, Chumash speaking peoples occupied a large area that extended south along the California coast from San Luis Obispo County into Los Angeles County and east to Kern County, and included the Santa Barbara Channel Islands of San Miguel, Santa Rosa, Santa Cruz, and Anacapa (Glassow 1980; Grant 1978). The project area lies within the territory occupied at that time by a native group speaking Ventureño, one of the six major dialects of the Chumash language.

Known as the Ventureño Chumash, this group was distinguished from their culturally similar neighbors to the west and north, the Ynezeño and Barbareño Chumash, on the basis of linguistic deviations noted by the early Spanish missionaries of the area, rather than by any apparent difference in social or economic organization. The Ventureño (so named because of their association with Mission San Buenaventura) were the southernmost of all the Chumash peoples and spoke one of six Chumashan dialects considered as forming a core group of more closely related forms (Grant 1978).

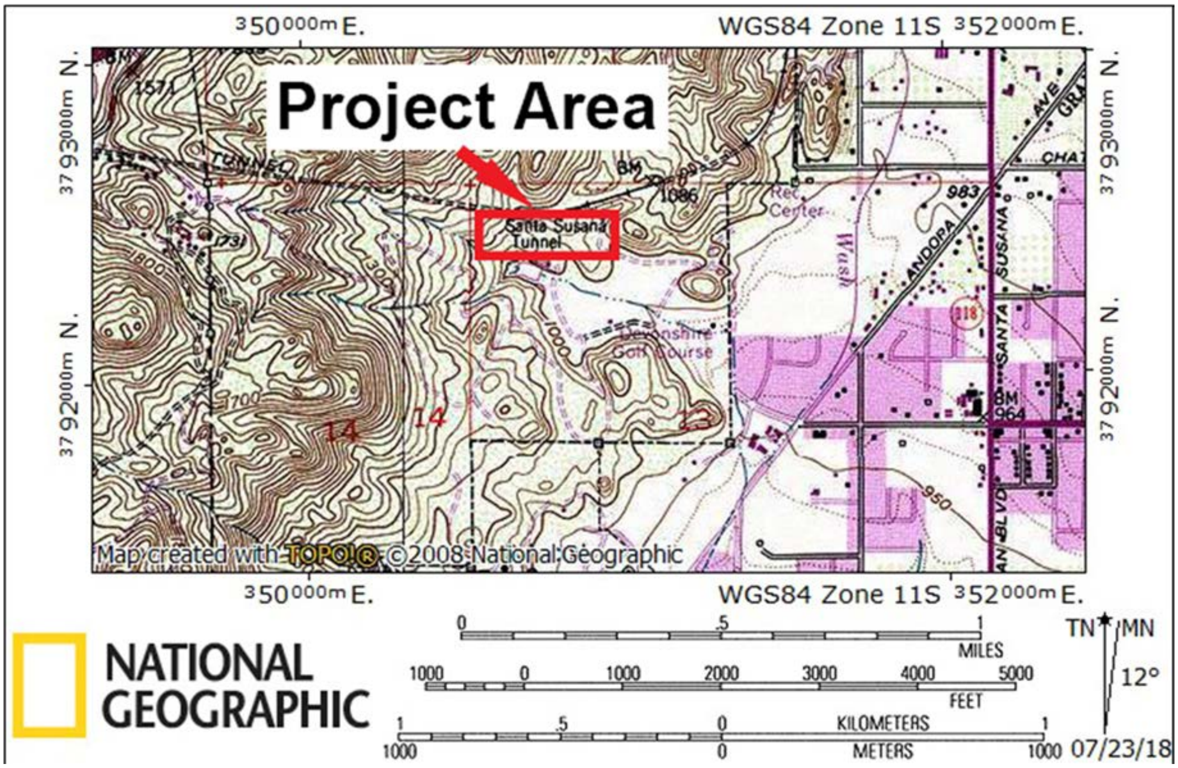

Figure 1. Vicinity Map, USGS Oat Mountain and Santa Susana, CA, 7.5 minute Quadrangles.

Native American culture in this region evolved over the course of at least 9,000 years and has been described as having achieved a level of social, political, and economic complexity not ordinarily associated with hunting and gathering groups (Greenwood and Browne 1969). Ethnographic information about the culture is most extensive for the coastal populations, and the culture and society have been well documented for groups such as the Barbareño and Ventureño Chumash. Much of what is known of the Ventureño has been provided by the journals of early Spanish explorers and by accounts of Chumash informants.

The Ventureño, like their neighbors, exploited a wide variety of marine and terrestrial resources within an ecosystem similar to that of their neighbors in Santa Barbara County. The limited area occupied by the Barbareño Chumash, a narrow coastal plain bounded on the north by the Santa Ynez Mountains, combined with a productive near shore fishery, resulted in the establishment of substantial permanent villages (Glassow and Wilcoxon 1979). These large villages provided centralized locations from which the inhabitants ventured to exploit available or seasonal resources and dispersed surplus resources and manufactured goods through intervillage exchange networks.

History

European incursions into the Ventureño area began with the arrival by sea of Juan Rodriguez Cabrillo on October 10, 1542, at the coastal Chumash village of *Shisholop*. Here, at the present site of the City of Ventura, the Spaniards were met by “many very good canoes, each of which held 12 or 13 Indians.” This prompted the visitors to name the settlement the Pueblo de las Canoas. Cabrillo and his men remained in the area until the 13th of the month, trading glass beads for items of local produce (Engelhardt 1930:4; Grant 1978:518). This first encounter was followed in December 1602 by a visitation of three ships under the command of Sebastian Vizcaino, and again in August 1769 by the land expedition by Gaspar de Portolá.

The Franciscan Padres Juan Crespi and Francisco Gomez accompanied the Portolá Expedition, and Crespi described the native “pueblo” as consisting of 30 large houses with no fewer than 400 inhabitants. The first Roman Catholic Mass was celebrated at this time, the location was renamed La Asuncion de Nuestra Senora, and the seeds of the coming Spanish mission system were planted in the local populace (Engelhardt 1930:6-10).

On Easter Sunday, March 31, 1782, Junipero Serra established the new “Mission of the Seraphic Doctor, San Buenaventura,” and left as its first residents Fr. Pedro Cambon and a small company of guards (Engelhardt 1930:16). The project area was within Mission San Buenaventura had primary jurisdiction. The introduction of the Spanish mission system into Ventureño territory brought about dramatic changes in the aboriginal way of life. Between the time of the establishment of the Mission San Buenaventura and that of Mexican independence and the secularization of the mission lands in 1834, ancient lifeways gradually began to disappear. Villages were abandoned, traditional marriage patterns were inhibited, hunting and gathering activities were disrupted as newly introduced agricultural practices altered the landscape, and large portions of the native population died from European diseases to which they lacked immunities.

Mission San Buenaventura flourished for nearly 50 years until a combination of factors led to its decline. The toll which introduced European diseases took on the neophyte population of native Chumash peoples, the waning financial support from Spain, and the eventual takeover by the newly established Mexican government in 1822, all weakened the entire mission system. The final blow came in 1833, when the Mexican government secularized the mission system. This action removed most of the mission property from the hands of the church and made it part of the public domain, available for lease or sale (Drapeau 1965). Perhaps to maintain the self-sufficient lifestyle of the mission, the church was allowed to keep, in addition to the church building itself, “... an enclosed garden of an area of about five hundred varas square more or less” (Drapeau 1965). The remainder of the vast mission tract was granted to José de Arnaz in 1846 and became the Ex-Mission Rancho (Drapeau 1965; Thompson and West 1883). The City of San Buenaventura was officially organized in 1866 encompassing lots in the immediate vicinity of the mission and dominated by non-Anglo inhabitants.

After the Treaty of Guadalupe Hidalgo in 1846, the Euroamericans took over California and declared that Governor Pio Pico did not have the authority to lease and sell mission lands.

The United States Lands Commission heard petitions for claims to mission lands and voided many of the transactions concluded under Pico's hegemony.

The Rancho Period has been romanticized in literature and film as a time of easy wealth and leisure notable for dashing horsemanship and Hispanic hospitality on a grand scale. The reality was the more prosaic work of making a living in the cattle business (Greenwood 1989:451-466). The discovery of gold in northern California created a boom in the cattle industry which fed the hordes of miners searching for gold. During the 1860s, the Euroamerican population grew rapidly, partly because many of the old rancho families lost title to their land, leaving a vacuum which was promptly filled by settlers from central and eastern United States.

In the 1860s homesteaders moved into Chatsworth and one of the initial families was Nels and Ann Johnson who homesteaded 160 acres beneath the Santa Susanna Pass (Roderick 2001:32). Chatsworth Railroad History begins in 1893 when the Southern Pacific completed what is known as the Burbank branch all the way to Chatsworth with a depot near the intersection of Topanga and Marilla. In 1898 an additional mile of track was added up through what is now the Oakwood Cemetery into the Chatsworth quarry, now a part of the Santa Susana Pass State Historic Park. The quarry sent sandstone boulders to a stone mill in Los Angeles to further shape and form the stone. They also delivered sandstone to San Pedro Harbor where they were used for the breakwater. In 1898, railroad construction began on a short-cut to Burbank from Ventura in what was called the Montalvo Cutoff. The most difficult work was encountered in the pass, where three separate tunnels were blasted for the most part out of solid rock. During that time, Chatsworth became a boom town, with many of the workers living in a “tent” city near the heading of the main tunnel. Although the listed resident population in Chatsworth is 23 in 1900, the tunnel construction brought in so many workers that by 1904 the Santa Susana School (now Chatsworth Park Elementary) at Devonshire and Topanga had 120 students (Vincent 2014).

Chatsworth Park South was closed in 2008 due to lead contamination. Contamination from lead bullets used in the 1950s and 1960s at a former gun club owned by actor Roy Rogers prompted the closure. Investigators discovered toxic soil contamination left over from shotgun pellets and clay pigeons used on its 12-acre skeet-shooting range.

West Valley Feeder No 1 is a concrete cylinder that conveys water to two agencies (Las Virgenes Municipal Water District and Calleguas Municipal Water District). The pipeline was constructed in 1962. West Valley Feeder No 1 was originally constructed by Calleguas Municipal Water District and originally named Calleguas Conduit Unit 4.

LITERATURE AND ARCHIVAL REVIEW

Record Search Summary: West Valley Feeder No. 1, Stage 3, MWD (Chatsworth)

RESULTS

Resources within Project Area: One, 19-150434 (1900 structure)

Site 19-150434 is the reported location of a ca. 1900 structure. The location was identified on the basis of a 1903 15 minute USGS quadrangle (Scale = 1:62,500 feet) and was not field verified at the time of recording (Edberg 1978). A Universal Transverse Mercator grid point was provided and compared with potential impact areas. Two of the contractor laydown areas on the east side of the project area are within approximately 300 feet of the reported location of the ca. 1900 structure.

Archaeological resources within search area (0.5 mi radius): 19

CA-LAN-448	CA-LAN-3498	CA-LAN-3579
CA-LAN-449	CA-LAN-3500	CA-LAN-120078
CA- LAN-640	CA-LAN-3505	CA-LAN-120084
CA-LAN-1028	CA-LAN-3506	CA-LAN-176735
CA-LAN-1126	CA-LAN-3507	
CA-LAN-2174	CA-LAN-3509	
CA-LAN-3494	CA-LAN-3512	

Three archaeological sites, CA-LAN-3507 (Mealey and Buxton 2004), CA-LAN-3512 (Mealey, Farmer, and Brodie 2005), and CA-LAN-120084 (Mealey, Farmer, and Brodie 2005) were recorded outside of and west of the western terminus of proposed project area, i.e., laydown areas, access road, and trail. The three sites are recorded between 450 feet and 1000 feet from the nearest portion of the project area. Two of the archaeological sites, CA-LAN-3507 and CA-LAN-3512), were identified as small dispersed flake scatters. The third site, CA-LAN-120084, consists of three mortared red bricks and a scattering of white quartz rocks.

Surveys/Reports including Project Area: None

Surveys/Reports within search area: 31

LA-81	LA-2252	LA-4123
LA-160	LA-2623	LA-4125
LA-397	LA-2645	LA-6599
LA-631	LA-2874	LA-7837
LA-853	LA-3009	LA-8255
LA-1015	LA-3185	LA-9070

LA-1050	LA-3340	LA-10569
LA-1051	LA-3452	LA-10637
LA-2002	LA-3487	LA-10651
LA-2079	LA-3499	LA11164
		VN-572

Historic Resources Inventory (HRI) results (0.5 mile search radius):

Evaluated Historical Resources: 1

- Old Santa Susana Stage Road

Local Historical Resources: 1

- City of Los Angeles Historic Cultural Monument No. 92, Old Stage Coach Trail Property (Old Santa Susana Stage Road), South Chatsworth Park

County Historical Resources: 1

- Ventura County Historical Landmark #104, Old Santa Susana Stage Road

California State Points of Historical Interest: None

California State Historical Landmarks: None

National Register of Historic Places Properties: 1

- Old Santa Susana Stage Road, Chatsworth, CA. NRHP Ref. No. 74000517, listed Oct. 1974.

Historic Maps:
1903 USGS Santa Susana, California, 15' quadrangle map.
This map depicts a segment of the Santa Susana Tunnel, which carried a Southern Pacific Railroad line through the Santa Susana Pass, along with an above-ground section of the rail line, running east-west across the northern boundary of the current subject property. Also, within the study area is the western end of an unimproved (dirt) road that appears to have been a northwesterly extension of Devonshire Street. Along this road, in the immediate vicinity of the project area, were at least two dwellings, with three additional dwellings in close proximity to the southeast. Also, within 0.25 mile of the subject property, directly to the south, was a mining property with one associated dwelling. An unimproved road that provided access to the mine extended to the southeast, and this route continued to the northwest where it is depicted as a 'trail.' There were two or three additional dwellings located within 0.5 mile of the subject property, located around the western terminus of

Figure 2. Project Impact Areas.

Devonshire Street. There were no additional historic features in the vicinity of the project area at this date.

1927 USGS Chatsworth, California, 6' quadrangle map.
This 1927 map only depicts the area east of the Los Angeles County line in detail. The unimproved road and dwellings that had been illustrated within and near the subject property on the 1903 map are no longer indicated. The railroad alignment remained along the north edge of the project area, and in addition to the segment of the Somis Branch of the Southern pacific Railroad and Santa Susana Tunnel, the only historic feature depicted within 0.5 mile of the project area is a single dwelling located along the south side of the tracks immediately east of the project areas.

1933 USGS Chatsworth, California, 6' quadrangle map.
Like the 1925 map, this map illustrates only a few features west of the Ventura/Los Angeles County line. The only historic feature shown in proximity to the project area is the Somis Branch of the Southern Pacific Railroad, along its northern boundary.

1940 USGS Chatsworth, California, 6' quadrangle map.
The 1940 map depicts the Southern Pacific Railroad alignment and the Santa Susana Tunnel along the northern edge of the subject property. To the south, the Oakwood Cemetery had been established, and several new unimproved roads are indicated immediately north of the cemetery, approximately 0.5 mile from the subject property. No other historic features are represented within the search area.

1943 USGS Santa Susana, California, 15' quadrangle map.
In addition to the railroad alignment and tunnel, this map illustrates a new westward extension of Devonshire Street that had been established within 0.30 mile south of the project area by this date. There were approximately eight new residences along this unimproved roadway. Additionally, a trail is depicted to the southwest of the project areas that followed the base of the hills roughly 0.25 mile away. There was no additional historic development in the vicinity of the subject property at the time.

1951 USGS Santa Susana, California, 7.5' quadrangle map.
This map illustrates the western quarter of the search area for the project. It shows no historic features within that section beyond the Southern Pacific Railroad alignment.

1952 USGS Oat Mountain, California, 7.5' quadrangle map.
This map illustrates that by 1952, Devonshire Street had been extended to the base of the foothills south of the subject property, and this street was now paved. There was an unimproved road that continued northward from the west end of Devonshire, and along this road were two new residences within 1000 feet of the project areas. Beyond the Southern Pacific rail alignment and tunnel, there are no other buildings or historic features indicated in the vicinity of the subject property.

1969 USGS Oat Mountain, California, 7.5' quadrangle map.

The 1969 quadrangle map indicates that the unimproved roadway depicted on the 1952 map extending northward from the west end of Devonshire Street has been further extended to the north, to the southern boundary of the project areas. One new dwelling had been constructed at the north end of this road, and there was a second new dwelling near the east project area boundary. This was accessed by another new unimproved road that approached from the east. Also depicted is the Devonshire Golf Club, located within 0.25 mile southeast of the project areas. There were no additional historic features located in proximity to the project areas.

1969 USGS Santa Susana, California 7.5' quadrangle map.

This map is identical to the 1951 Santa Susana quadrangle map and depicts no historic features within this section of the search area beyond the Southern Pacific Railroad alignment.

Sanborn Map Co. Insurance Maps

There are no Sanborn insurance maps that include any portion of the record search area.

SURVEY RESULTS

The field survey was conducted on June 5 and 6, 2018 by John M. Foster, RPA. Visibility within the project area was generally poor with dense vegetation and steep slopes hindering observations of the ground surface. However, most of the impact areas (Figure 2) had excellent visibility, except for the proposed access road alignment, depicted in red on Figure 2. Transects with 10 meter spacing were conducted over each impact area.

Due to limited ground visibility in the western part of the project area, proposed alignment, it could not be determined if archaeological resources were present (Figure 2). The location of the ca. 1900 (19-150434) structure was carefully transected and no evidence of a structure was found. The scale of a 15 minute map makes precise locations difficult to determine and it likely that 19-150434 (1900 structure) is in the area but not in any of the proposed impact areas for this project.

It is evident from the closest recorded archaeological sites (dispersed flake scatters) that it is likely that additional flakes can be found under ideal conditions.

IMPACTS

Due to the limited ground visibility impacts to potential archaeological resources could not be determined for the proposed alignment. No archaeological resources were observed in the other impact areas.

RECOMMENDATIONS

The proximity of recorded archaeological resources coupled with poor ground visibility in some areas warrants a recommendation for monitoring by an archaeological and Native American monitor. Excavation strategies to determine if resources are present is not recommended since the closest archaeological sites consist of dispersed flake scatters and are not likely to be identified during the testing process. It is our opinion that monitoring would be the most effective means to identify cultural resources in the project areas.

In the event of an accidental discovery of any human remains in a location other than a dedicated cemetery, the steps and procedures specified in Health and Safety Code 7050.5, State CEQA Guidelines 15064.5(d), and Public Resources Code 5097.98 shall be implemented. Specifically, in accordance with Public Resources Code (PRC) Section 5097.98, the Los Angeles County Coroner shall be notified within 24 hours of the discovery of potentially human remains. The Coroner typically would then determine within two working days of being notified if the remains are subject to his or her authority. If the Coroner recognizes the remains to be Native American, he or she would contact the Native American Heritage Commission (NAHC) by phone within 24 hours, in accordance with PRC Section 5097.98. The NAHC typically would then designate a Most Likely Descendant (MLD) with respect to the human remains within 48 hours of notification.

The MLD typically would then have the opportunity to recommend to the property owner or the project proponent means for treating or disposing of, with appropriate dignity, the human remains and associated grave goods within 24 hours of notification. Whenever the NAHC is unable to identify a MLD, or the MLD fails to make a recommendation, or the landowner or his or her authorized representative rejects the recommendation of the MLD and the mediation provided for in subdivision (k) of PRC Section 5097.94 fails to provide measures acceptable to the landowner, the landowner or his or her authorized representative would re-inter the human remains and items associated with Native American burials with appropriate dignity on the property in a location not subject to further subsurface disturbance.

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APPENDIX E

ENERGY ANALYSIS

Energy Use Summary

Construction Phase (gallons/construction perioc	**Gasoline**	**Diesel**
Construction Vehicles	0	5,148
Worker Trips	611	2
Vendor Trips	55	1
Haul Trucks	0	74
Total	666	5,226

Operations Phase (gallons/year)		**Gasoline**	**Diesel**	**Natural Gas (kBTU/yr)**	**Electricity (kWh/yr)**
Hotel		0	0	9,590,000	2,531,200
	0	0	0	0	0
	0	0	0	0	0
All Land Uses		0	0	9,590,000	2,531,200

Construction Offroad Equipment Fuel Use

PhaseName	OffRoadEquipmentType	OffRoadEq	UsageHours	HorsePower	Load Factor	Horsepower Category	Num Days	Year	Fuel Consumption Rate (gal/hour)	Fuel Type	Total Fuel Consumption (gal/construction period)
Demolition	Concrete/Industrial Saws	0	8	81	0.73	100	22	2019	4.7	Gasoline	0
Demolition	Rubber Tired Dozers	0	1	247	0.4	300	22	2019	4.5	Diesel	0
Demolition	Tractors/Loaders/Backhoes	1	6	97	0.37	100	22	2019	1.6	Diesel	78
Site Preparation	Excavators	1	8	158	0.38	175	21	2019	2.9	Diesel	184
Site Preparation	Graders	0	8	187	0.41	175	21	2019	3.1	Diesel	0
Site Preparation	Tractors/Loaders/Backhoes	0	8	97	0.37	100	21	2019	1.6	Diesel	0
Grading	Concrete/Industrial Saws	0	8	81	0.73	100	44	2019	4.7	Gasoline	0
Grading	Cranes	1	8	231	0.29	300	44	2019	3.3	Diesel	337
Grading	Excavators	1	8	158	0.38	175	44	2019	2.9	Diesel	386
Grading	Graders	1	8	187	0.41	175	44	2019	3.1	Diesel	454
Grading	Rubber Tired Dozers	0	1	247	0.4	300	44	2019	4.5	Diesel	0
Grading	Tractors/Loaders/Backhoes	1	6	97	0.37	100	44	2019	1.6	Diesel	155
Building Construction	Cranes	1	4	231	0.29	300	109	2019	3.3	Diesel	417
Building Construction	Excavators	1	8	158	0.38	175	109	2019	2.9	Diesel	956
Building Construction	Forklifts	0	6	89	0.2	100	109	2019	2.0	Diesel	0
Building Construction	Rubber Tired Dozers	1	8	247	0.4	300	109	2019	4.5	Diesel	1,555
Building Construction	Tractors/Loaders/Backhoes	1	8	97	0.37	100	109	2019	1.6	Diesel	513
Paving	Cement and Mortar Mixers	0	6	9	0.56	25	22	2019	0.4	Gasoline	0
Paving	Pavers	1	7	130	0.42	100	22	2019	1.7	Diesel	113
Paving	Rollers	0	7	80	0.38	100	22	2019	1.7	Diesel	0
Paving	Tractors/Loaders/Backhoes	0	7	97	0.37	100	22	2019	1.6	Diesel	0

Total		**5,148**
Total	**Gasoline**	**-**
Total	**Diesel**	**5,148**

Construction Phase - Onroad Energy Use

Year 2020

Vehicle Types	MPG by Fuel Type			Population by Fuel Type			
	GAS	DSL	ELEC	GAS	DSL	ELEC	Total
LDA	29.3	46.3		6,343,244	51,116	90,986	6,394,359
LDT1	25.2	22.1		692,885	447	2,466	693,332
LDT2	23.0	33.7		2,169,628	11,368	12,535	2,180,995
LHDT1	10.3	21.0		178,175	106,680		284,856
LHDT2	9.0	19.0		29,750	41,895		71,645
MCY	36.5			276,048			276,048
MDV	18.8	25.9		1,557,729	27,452	3,954	1,585,180
MH	5.0	10.4		36,101	12,007		48,108
MHDT	5.0	10.1		25,210	120,277		145,487
HHDT	3.9	6.4		88	103,820		103,908
OBUS	4.9	8.1		5,971	4,179		10,150
SBUS	9.0	7.4		2,328	6,543		8,871
UBUS	4.8	6.3		938	18	17	956

Input							Gasoline Consumption			Diesel Consumption		
Phase Name	Worker Trip Number	Vendor Trip Number	Hauling Trip Number	Worker Trip Length	Vendor Trip Length	Hauling Trip Length	Worker	Vendor	Haul	Worker	Vendor	Haul
Demolition	3	0	2	14.7	6.9	20						
Site Preparation	3	0	18	14.7	6.9	20						
Grading	10	0	4	14.7	6.9	20						
Building Construction	3	1	0	14.7	6.9	20						
Paving	3	2	0	14.7	6.9	20						
Adjusted												
Demolition	66	0	2	14.7	6.9	20	42	0	0	0	0	6
Site Preparation	63	0	18	14.7	6.9	20	40	0	0	0	0	56
Grading	440	0	4	14.7	6.9	20	279	0	0	1	0	12
Building Construction	327	109	0	14.7	6.9	20	208	39	0	1	1	0
Paving	66	44	0	14.7	6.9	20	42	16	0	0	0	0
Total							**611**	**55**	**0**	**2**	**1**	**74**

APPENDIX F

REPORT OF GEOTECHNICAL STUDY

REPORT OF GEOTECHNICAL STUDY
WEST VALLEY FEEDER 1 ACCESS ROADS AND VALVE IMPROVEMENTS
WIDENING PROJECT
CHATSWORTH, CALIFORNIA

MAY 15, 2018

May 15, 2018
Kleinfelder Project No. 20180213.002A

Mr. Bei Su, PE
Metropolitan Water District of Southern California
700 North Alameda Street
Los Angeles, California 90012

SUBJECT: Final Report of Geotechnical Study
West Valley Feeder 1 Access Roads and Valve Improvements
Chatsworth, California

Dear Mr. Su:

Kleinfelder is pleased to present this report summarizing our geotechnical investigation for the subject project. The purpose of our geotechnical investigation was to evaluate subsurface conditions and provide geotechnical recommendations for the design and construction of the proposed project. The conclusions and recommendations presented in this report are subject to the limitations presented in Section 6.

We appreciate the opportunity to provide geotechnical engineering services to you on this project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned at 951.801.3681.

Sincerely,

KLEINFELDER WEST, INC.

REGISTERED PROFESSIONAL ENGINEER JEFFERY D. WALLER NO. 2947 EXP. 9-30-18 GEOTECHNICAL STATE OF CALIFORNIA

Jeffery D. Waller, PE, GE
Senior Geotechnical Engineer

PROFESSIONAL GEOLOGIST MICHAEL O. COOK CEG #1716 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

Michael O. Cook, PG, CEG
Senior Engineering Geologist

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1 INTRODUCTION

Kleinfelder performed a geotechnical study for Metropolitan Water District of Southern California (MWD) for the proposed project in Chatsworth, California. This report summarizes the results of our field exploration, laboratory testing, and engineering analysis and provides recommendations for design and construction for the subject project. The approximate location of the project presented in this report is shown on Figure 1, Site Vicinity Map. The purpose of our geotechnical study was to evaluate subsurface soil conditions and provide geotechnical recommendations for the design and construction of the proposed project. The scope of our services was presented in our proposal dated December 1, 2017.

Our report includes a description of the work performed, a discussion of the geotechnical conditions observed at the site, and recommendations developed from our engineering analyses of field and laboratory data.

1.1 PROJECT DESCRIPTION

We understand the proposed project includes improvements to manholes and valve structures along the West Valley Feeder No. 1, and construction of two new access roads to provide maintenance access to the pipeline and valve structures. The roads are proposed to be constructed with Portland cement concrete (PCC) and may have sections where asphaltic concrete (AC) is used. On each alignment, a concrete Arizona crossing is also proposed at the location where the access roads cross the existing seasonal creeks.

Preliminary Plan and Profile documents for the project were reviewed in preparation of this report. The location of the proposed alignment selected by MWD are shown on Figure 2, Field Exploration Location Map. The proposed alignments may have small retaining walls. In steep sections of the roadway, concrete keys are proposed beneath the pavement to reduce the potential for sliding of the pavement.

1.2 SCOPE OF SERVICES

The scope of our geotechnical study consisted of a literature review, site reconnaissance, subsurface explorations, geotechnical laboratory testing, engineering evaluation and analysis, and preparation of this report. A description of our scope of services performed for the geotechnical portion of the project follows.

Task 1 – Background Data Review. We reviewed readily-available published and unpublished geologic literature in our files and the files of public agencies, including selected publications prepared by the California Geological Survey, California Division of Mines and Geology, and the U.S. Geological Survey. We also reviewed readily available seismic and faulting information, including data for designated earthquake fault zones as well as our in-house database of faulting in the general site vicinity.

Task 2 – Field Exploration. On June 19, 2017, representatives of Kleinfelder and MWD met at the project site to perform reconnaissance of the proposed alignments and the current conditions. Each of the proposed alignments and many of the valve structures to be reconstructed were observed as well.

Kleinfelder supervised exploration of 5 hollow stem auger borings. The approximate locations of the borings are presented on Figure 2, Field Exploration Location Map. The borings were drilled to provide general information in order to characterize subsurface materials and perform our analyses.

Prior to beginning subsurface exploration, each of the 5 boring locations were marked and Kleinfelder notified Underground Service Alert (USA) of our intent to dig in accordance with California State law.

All exploratory borings were drilled and logged on January 30, 2018. The borings were advanced to depths ranging from approximately 11½ to 21½ feet below the existing ground surface (bgs) using a limited access track-mounted drill rig operated by 2R Drilling of Chino, California. Bulk and drive samples were retrieved from the borings, sealed and transported to our laboratory for further evaluation. A staff professional of Kleinfelder supervised the sampling, logged and visually classified the excavated soil cuttings and samples retrieved. Bulk soil samples were generally collected within the upper 5 feet of each boring and drive samples were collected at approximate 5-foot intervals using split-spoon samplers. With the exception of Boring B-3, the excavated soil cuttings were used to backfill the excavations. Boring B-3 was backfilled with a cement/bentonite grout due to concerns of potential load contamination due to being located near a previous shooting range. The Logs of Borings B-1 through B-5 are included in Appendix A, Field Explorations at the end of this report. The approximate locations of the borings are shown on Figure 2, Field Exploration Location Map.

On January 11, 2018, two Seismic Refraction Surveys were performed at the site by Advanced Geoscience Inc. (AGI) and their approximate locations are shown on Figure 2. AGI completed their field work and processed the data using the RAYFRACT program to prepare scaled, 2D elevation profiles of the seismic compressional-wave velocity layering. The Summary Report prepared by AGI is presented in Appendix C, Seismic Refraction Survey Report.

Task 3 – Laboratory Testing. Laboratory testing was performed on selected samples to provide parameters for engineering evaluation. Testing consisted of in-situ density and moisture content, sieve and hydrometer, direct shear, expansion index, maximum density and optimum moisture, R-value, and Preliminary Corrosion Potential. Descriptions of the laboratory tests performed and the results of the testing are presented in Appendix B, Laboratory Testing.

Task 4 – Geotechnical Analyses. Field and laboratory data were analyzed in conjunction with our understanding of the proposed project from the referenced MWD Civil Drawings to provide geotechnical recommendations for the design and construction of the proposed access roads and valve structure improvement. Seismic parameters presented are based on the 2016 California Building Code (CBC).

Task 5 – Report Preparation. This report summarizes the work performed, data acquired, and our findings, conclusions, and geotechnical recommendations for the design and construction of the proposed improvements. The report includes the following items:

- Site location map and site plan showing the approximate boring locations;
- Logs of borings (Appendix A);
- Results of laboratory tests (Appendix B);
- Seismic Refraction Survey Summary Report by AGI (Appendix C);
- Discussion of general site conditions;
- Discussion of general subsurface conditions as encountered during field exploration;
- Discussion of regional and local geology and site seismicity;

- Discussion of geologic and seismic hazards;
- Recommendations for site preparation, earthwork, temporary slope inclinations, fill placement, and compaction specifications, including excavation characteristics of subsurface soil deposits;
- Recommendations for retaining wall foundation design, allowable bearing pressures, and embedment depths;
- Recommendations for seismic design parameters in accordance with the 2016 CBC;
- Preliminary slope stability conclusions for Cross Section C, WVF1 Station 1415+42 access road section at Station 1+50 for Option 1, presented on the MWD Civil Drawings; and
- Preliminary slope stability conclusions for Cross Section F, WVF1 Station 1416+33 access road section at Station 2+20 for Option 2, presented on the MWD Civil Drawings.

2 SITE DESCRIPTON

2.1 SITE DESCRIPTION

The project site is located in the Chatsworth area of the City of Los Angeles, California. Chatsworth Park South bounds the site on the south and east sides. Hillside areas with local rugged rock outcrops, intervening drainage channels, and local dense vegetation bound the access road locations on the north and west sides. The southern and eastern portions of the access road locations are low-lying areas with sparse vegetation. Surface water was observed flowing within one of the drainage channels during the June 19, 2017, site visit. The channel is located at approximate Station 0+68 as shown on the referenced MWD Civil Drawings (MWD, 2018).

3 GEOLOGY

3.1 REGIONAL GEOLOGIC SETTING

The site is located within the western Transverse Ranges geomorphic province (Norris and Webb, 1990). The Transverse Ranges province is characterized by roughly east-west trending, convergent structural features in contrast to the predominant northwest-southeast structural trend of Coast Ranges and Peninsular geomorphic provinces in California (CGS, 2002). The Transverse Ranges province's east-west trending folds and faults are due to north-south tectonic compression from movement along the San Adreas fault system, resulting in one of the most seismically active regions in California. The western Transverse Ranges extends generally from the Los Angeles/San Bernardino County line on the east to Point Arguello west of Santa Barbara.

Structurally, the portion of the western Transverse Ranges where the project site is situated is bounded on the north by the Sierra Madre fault zone – San Fernando section and the Santa Monica Mountains to the south.

The primary geologic unit comprising the foothills of the project area is the Upper Cretaceous Chatsworth Formation. The Chatsworth Formation is a turbidite sequence of marine fan deposits composed primarily of arkosic sandstones (Link et al., 1984) with lesser siltstones and conglomerates interbedded with shales (Cilona et al., 2016). Young alluvial fan deposits underly the San Fernando Valley east of the project site. The geologic units are presented on Figure 3, Regional Geologic Map.

3.2 SUBSURFACE CONDITIONS

Subsurface conditions at the project site consist of young alluvial deposits overlying bedrock of the Cretaceous-age Chatsworth Formation. On January 30, 2018, Kleinfelder drilled five borings to a maximum depth of 21.5 feet below ground surface.

The following is a general description of the subsurface conditions and the bedrock characteristics that can be applied to subsurface conditions at the locations explored. Subsurface materials encountered at the locations explored generally consisted of a thin veneer of artificial fill or native young alluvium overlying bedrock of the Chatsworth Formation. Detailed descriptions of the deposits are provided in our logs of borings presented in Appendix A.

3.2.1 Fill and Native Soils

Fill and alluvial soils encountered generally consisted of medium dense to dense silty sand to sand with gravel and some sandy clay. These soils were generally present locally within the upper 3 to approximately 5 feet except in B-3, where it extended to 16.5 feet (maximum depth explored). Laboratory testing of two bulk samples of subgrade soils collected at borings B-3 and B-5 resulted in R-values of 19 and 29, respectively. Laboratory dry density in boring B-3 of the native soil was approximately 113 pounds per cubic-foot (pcf) with a moisture content of approximately 6.7 percent.

3.2.2 Bedrock

Bedrock is predominantly comprised of a fine-grained yellow-brown sandstone of the Chatsworth Formation. The bedrock is thickly-bedded (3-10 feet thick) and uniformly dip to the northwest between approximately 10 and 15 degrees. Bedrock materials encountered below native and fill soils were consistent with Chatsworth Formation with blow counts greater than 50 for 6 inches. Laboratory dry densities of samples with bedrock materials ranged from approximately 98 to 118 pounds per cubic-foot (pcf). Laboratory moisture contents ranged from approximately 3.6 to 12.2 percent.

3.3 GROUNDWATER

Groundwater was not encountered in any of the borings performed at the site on January 30, 2018. There are no known active groundwater wells or monitoring wells on or within near proximity to the project site. Since the sites elevation is approximately 50 to 110 feet higher than the general ground surface of the San Fernando Valley located to the east, we do not anticipate encountering groundwater in areas underlain by shallow bedrock. Although not encountered in the borings, shallow perched groundwater could occur in areas underlain by alluvium.

Fluctuations of the groundwater level, localized zones of perched water, and variations in soil moisture content should be anticipated during and following the rainy season (late fall to early spring). Irrigation of landscaped areas on and adjacent to the site can also cause a fluctuation of local groundwater levels.

3.4 FAULTING

There is a high potential for moderate to strong seismic activity to occur during the design life of the project. The site is in the highly seismic Southern California region within the influence of several fault systems that are considered to be active or potentially active. An active fault is defined by the State of California as being a “sufficiently active and well defined fault” that has exhibited surface displacement within Holocene time (about the last 11,000 years). A potentially active fault is defined by the State as a fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago). These active and potentially active faults are capable of producing potentially damaging seismic shaking at the site. It is anticipated that the project site will periodically experience ground acceleration as the result of earthquakes. Active faults without surface expression (blind faults) and other potentially active seismic sources, which are capable of generating earthquakes, are not currently zoned and are known to be locally present under the region.

The site is not located within a State of California Earthquake Fault Rupture Hazard Zone (Bryant and Hart, 2007, CGS, 2017). Based on our geologic literature review, no mapped active or potentially active fault traces are known to transect the project site (Treiman, 2000). The closest active fault to the site is the Sierra Madre fault Zone – Santa Susana and San Fernando sections faults located approximately 7.0 miles and 7.5 miles, respectively from the site (Barrows et al., 1975).

3.5 SEISMIC HAZARD ZONES

The project site is not located within a State of California designated area with potential liquefaction or earthquake-induced landslide zones (CGS, 2017). See Section 4.2.1 for the results of our liquefaction analysis at the site.

Landslides are ground failures (several tens to hundreds of feet deep) in which a (mass of earth material, including debris and often portions of bedrock) large section of a slope detaches and slides downhill. Landslides are not to be confused with minor surficial slope failures (slumps), which are usually limited to the topsoil zone and can occur on slopes composed of almost any geologic material. Landslides can cause damage to structures both above and below the slide mass.

Structures above the slide area are typically damaged by undermining of foundations. Areas below a slide mass can be damaged by being overridden and crushed by the failed slope material.

Several factors can increase the potential for landsliding; slope angle, rock or soil type, bedding and foliation orientation, persistence of fractures, fracture density, zones of shearing or faulting, weathering, clay content, seismicity, water content, groundwater and the presence or absence of vegetation.

Although the area of the project site is not identified as a landslide hazard zone, some of these risk factors for landslides do exist at the site including: sloping terrain, the presence of nearby active faults, and historic seismic shaking.

3.6 FLOOD HAZARD

The Federal Emergency Management Agency (FEMA) maintains a collection of Flood Insurance Rate Maps (FIRM), which cover the entire United States. These maps identify those areas which may be subjected to 100 year and 500-year cycle floods. Based on our review of FEMA map panel 1040F (FEMA, 2008) the elevated portions of the site are situated within Zone D area in which flood hazards are undetermined, but possible. The southernmost portion of the project site is located within Zone A where there is a 1% annual chance of flood (100-year flood). No Base Flood Elevations are determined. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

3.7 EXPANSIVE SOILS

Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, utility leakage, perched groundwater, drought, or other factors and may cause unacceptable settlement or heave of pavements, sidewalks, curbs, gutters and other structures supported over these materials. The soils generally encountered during our study were granular and based on the Expansion Index test performed, they have a low to medium expansion potential.

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our field exploration, laboratory testing and engineering analyses conducted during this study, it is our professional opinion that the proposed project is geotechnically feasible, provided the recommendations presented in this report are incorporated into the project design and construction. The primary geotechnical considerations for site development are the presence of bedrock, stability of proposed slope cuts, and construction of pavement on a relatively steep grade.

The following opinions, conclusions, and recommendations are based on the properties of the materials encountered in the borings, the results of the laboratory-testing program, and our engineering analyses performed. Our recommendations regarding the geotechnical aspects of the design and construction of the project are presented in the following sections.

4.2 SEISMIC DESIGN CONSIDERATIONS

It is our understanding that after January 1, 2017, jurisdictional agencies review of proposed development will be based on the 2016 California Building Code (CBC). According to the 2016 CBC, every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7-10 (ASCE, 2010), excluding Chapter 14 and Appendix 11A. The seismic design category for a structure may be determined in accordance with Section 1613 of the 2016 CBC or ASCE 7-10. Based on the subsurface conditions encountered, the site can be classified as Site Class C. We have assumed that proposed structures will have a period of less than ½ second. This assumption should be verified by the project structural engineer.

The 2016 CBC seismic design parameters for the proposed access roads are summarized in Table 1.

Table 1

2016 CBC Seismic Design Parameters*

Site Class	C
Risk Category	I, II, and III
S_s (Figure 1613.3.1(1)) (g)	2.184
S_1 (Figure 1613.3.1(2)) (g)	0.695
F_a (Table 1613.3.3(1))	1.0
F_v (Table 1613.3.3(2))	1.3
S_{MS} (Equation 16-37) (g)	2.184
S_{M1} (Equation 16-38) (g)	0.904
S_{DS} (Equation 16-39) (g)	1.456
S_{D1} (Equation 16-40) (g)	0.603
PGA_M (ASCE 7-10 Equation 11.8-1) (g)	0.815

*Section references above are to the 2016 CBC unless otherwise noted.

4.2.1 Liquefaction

The term liquefaction describes a phenomenon in which saturated, cohesionless soils temporarily lose shear strength (liquefy) due to increased pore water pressures induced by strong, cyclic ground motions during an earthquake. Structures founded on or above potentially liquefiable soils may experience bearing capacity failures due to the temporary loss of foundation support, vertical settlements (both total and differential), and undergo lateral spreading. The factors known to influence liquefaction potential include soil type, relative density, grain size, confining pressure, depth to groundwater, and the intensity and duration of the seismic ground shaking. The cohesionless soils most susceptible to liquefaction are loose, saturated sands and some silt.

Based on the properties of the soils encountered in our test borings and our knowledge of geologic conditions in the area of the site, a site class of ‘C’ is considered appropriate as determined from Table 1613.5.2 of the 2016 California Building Code. The characteristics of the soil/bedrock, and depth to groundwater indicate that the site soils have a remote potential for liquefaction during a design-level earthquake.

4.3 EARTHWORK

Site preparation and earthwork operations should be performed in accordance with applicable codes, safety regulations and other local, state or federal specifications, and the recommendations included in this report. References to maximum unit weights are established in accordance with the latest version of ASTM Standard Test Method D1557. The earthwork operations should be overseen by a professional engineer from Kleinfelder.

4.3.1 Site Preparation

Existing pavements, utilities and other abandoned improvements should be demolished and removed from the site. All debris produced by demolition operations, including wood, steel, piping, plastics, etc., should be separated and disposed off-site. Existing abandoned utility pipelines which extend beyond the limits of the proposed construction and are to be abandoned in place, should be plugged with cement grout to prevent migration of soil and/or water. Demolition, disposal and grading operations should be overseen by a professional engineer from Kleinfelder.

Prior to general site grading, existing vegetation, organic topsoil, debris, and oversized materials (greater than 6 inches in maximum dimension) should be stripped and disposed outside the construction limits. Deeper stripping or grubbing may be required where higher concentrations of vegetation are encountered during site grading. The stripping work should include the removal of existing fill embankments, undocumented fill, and topsoil that, in the judgment of the geotechnical engineer, is compressible or contains significant voids. The stripping operation must expose a firm, non-yielding subgrade, or competent bedrock that is free of large voids. Stripped topsoil (less any debris) may be stockpiled and reused for landscaping purposes; however, this material should be evaluated for suitability if it is desired to use this material for engineered fill below structures.

Grading operations during the wet season or in areas where the soils are saturated may require significant provisions for drying of soils prior to compaction. If the project necessitates fill placement and compaction in wet conditions, we can provide alternatives for drying the soil.

Conversely, additional moisture may be required during the dry months. A sufficient water source should be available to provide adequate water during compaction. During dry months, moisture conditioning of the subgrade soils may be required if left exposed for greater than a few days.

4.3.2 Overexcavation

Organic, inert and oversized materials (greater than 6 inches in maximum dimension) should be stripped and isolated prior to removal of reusable soils. Pavement should be stripped and disposed off-site. Overexcavation should remove any loose or soft earth materials until a firm, relatively unyielding subgrade or competent bedrock is exposed, free of significant voids and organics. The subgrade soils exposed at the bottom of overexcavation should be observed or overseen by a professional engineer from our office prior to the placement of any fill. Prior to the placement of engineered fill, after site preparation, the bottom of the overexcavations should be proof-rolled and compacted to at least 90 percent relative compaction to the satisfaction of the geotechnical engineer-of-record. Additional removals, scarification and drying operations, and/or subgrade reinforcement may be required to stabilize soft, yielding subgrades.

The grading contractor should anticipate that additional processing and moisture conditioning of the onsite soils will be necessary during site grading to obtain material which is acceptable to be placed as engineered fill, as described in this report. The moisture conditioning of some of the soils will require significant drying and some soils will require the addition of moisture. These conditions could hamper equipment maneuverability and efforts to compact site soils to the recommended compaction criteria. Disking to aerate, chemical treatment, replacement with drier material, stabilization with a geotextile fabric or grid, or other methods may be required to mitigate the effects of excessive soil moisture and facilitate earthwork operations.

The grading contractor should also anticipate encountering oversized material greater than 6 inches in maximum dimension during excavation. Quantifying the actual amount of oversize material that could be encountered requires additional study.

Overexcavation of Pavements and Areas to Receive Fill: Pavements and areas to receive fill should be underlain by at least 2 feet of engineered fill. We recommend that overexcavation for pavements extend at least 2 feet below the bottom of pavement section and at least 2 feet below existing grade and proposed finished subgrade elevations. The 2 feet of overexcavation may be performed by overexcavating 18 inches of soil and scarifying, moisture conditioning, and compacting the bottom 6 inches of the excavation. Where the existing fill is deeper than 2 feet below bottom of pavement subgrades, we recommend that the overexcavation be deepened to remove existing fill soils.

We understand that reinforced concrete keys are proposed to be placed beneath the pavement in the steeper area of the proposed roadway. Due to the depth of the key, we anticipate that the excavation will extend into the competent bedrock. However, once excavated, the material at the bottom of the key should be evaluated by a representative of the Geotechnical Engineer of Record and may need to be extended deeper if unsuitable soils or unsuitable bedrock are encountered. If the excavation is extended, MWD may select to extend the key deeper with concrete or backfill the overexcavated area with engineered fill in accordance with the Engineered Fill section below.

On the downhill side, engineered fill should extend to the bottom of the key. The engineered fill should extend at least 2 feet laterally from the key and be placed as described below in the Engineered Fill section.

4.3.3 Scarification and Compaction

Following site stripping and any required grubbing and/or overexcavation, in areas to receive engineered fill that are not in competent bedrock should be scarified to a minimum depth of 8 inches, uniformly moisture-conditioned to a moisture content to near the optimum moisture content and compacted to at least 90 percent of the maximum dry density obtained using ASTM (American Society for Testing and Materials) Test Method D1557.

4.3.4 Rippability

The excavation and rippability of the existing bedrock was evaluated by performance of a seismic refraction survey. We have included the Summary Report as Appendix C of this report.

4.3.5 Engineered Fill

We anticipate that most of the on-site soils may be reusable as engineered fill once debris and oversized materials greater than 6 inches in diameter have been removed, and after any vegetation and organic debris is cleared and disposed off site. Fill should be placed in lifts no greater than 8 inches thick, loose measurement, and should be compacted to at least 90 percent of the maximum dry density. The moisture content of the soil should be within approximately 0 to 3 percent above the optimum moisture content. Any imported fill materials to be used for engineered fill should be sampled and tested for approval by the geotechnical engineer prior to being transported to the site. In general, well-graded mixtures of gravel, sand and non-plastic silt are acceptable for use as import fill.

Engineered fill should be compacted to at least 90 percent of maximum dry density obtained by the ASTM D1557 method of compaction with the upper 6 inches below pavements and structures compacted to at least 95 percent relative compaction.

In areas where the site needs to be raised in elevation per the MWD civil drawings, prior to the placement of engineered fill, the upper 24 inches below the existing site grade of the existing soils should be overexcavated and replace with engineered fill.

4.3.6 Temporary Excavations

All excavations must comply with applicable local, state, and federal safety regulations including the current OSHA Excavation and Trench Safety Standards. Construction site safety generally is the sole responsibility of the Contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. We are providing the information below solely as a service to our client. Under no circumstances should the information provided be interpreted to mean that Kleinfelder is assuming responsibility for construction site safety or the Contractor's activities; such responsibility is not being implied and should not be inferred.

The borings were advanced using a track-mounted, hollow-stem auger drill rig. Drilling was completed with moderate effort through the existing soil deposits and moderate to difficult drilling in the bedrock. Conventional earth moving equipment, as presented in the AGI report in Appendix C, should be capable of performing the excavations required for site development.

Near-surface soils encountered during our field investigation consisted predominantly of silty sand, clayey sand, and sand with silt. In our opinion, the soil encountered in our borings would be considered a Type 'C' soil with regard to the OSHA regulations. For this soil type, OSHA requires a maximum slope inclination of 1.5:1 (H:V) or flatter for excavations 20 feet or less in depth. Bedrock, due to its weathered condition, may be considered as a Type ‘B’ soil type with respects to OSHA regulations. Steeper cut slopes may be utilized for excavations less than 5 feet deep, depending on the strength, moisture content, and homogeneity of the soil/bedrock as observed during construction.

4.3.7 Pipe Bedding and Trench Backfill

If required, pipe bedding and pipe zone material should consist of sand or similar granular material having a minimum sand equivalent value of 30. The sand should be placed in a zone that extends a minimum of 6 inches below and 6 inches above the pipe for the full trench width. The bedding material should be compacted to a minimum of 90 percent of the maximum dry density or to the satisfaction of the geotechnical engineer's representative observing the compaction of the bedding material. Bedding material should consist of sand, gravel, crushed aggregate, or native free-draining granular material with a maximum particle size of 3/4 inch. Bedding materials should also conform to the pipe manufacturer's specifications, if available. Trench backfill above bedding and pipe zone materials may consist of approved, on-site or import soils placed in lifts no greater than 8 inches loose thickness and compacted to 90 percent of the maximum dry density based on ASTM Test Method D1557. Jetting of backfill is not recommended.

4.3.8 Stockpiling Excess Material

All stockpiles of excess soil materials should be kept away from the top of the excavations a minimum distance equal to the depth of the excavation. We recommend that stockpiles be constructed with a slope ratio of at least 2:1 (horizontal to vertical) and compacted to at least 85 percent relative compaction. The height of stockpiles should not exceed 10 feet. Compaction requirements and slope ratios are provided only for temporary stockpiling considerations, such as erosion control and temporary influences on excavations. We have not considered any long-term or structural support usage of stockpiles.

TEMPORARY SHORING

General

Temporary shoring may be required in areas adjacent to existing structures or improvements where excavations cannot be adequately sloped. Temporary shoring may consist of a turn-key shoring system, soldier piles and lagging, or other system. Recommendations for design of temporary shoring are presented below.

The shoring design should be provided by a civil engineer registered in the State of California and experienced in the design and construction of shoring under similar conditions. Once the final excavation and shoring plans are complete, the plans and design should be reviewed by Kleinfelder for conformance with the design intent and geotechnical recommendations provided herein.

Lateral Pressures

For the design of cantilevered shoring, an equivalent fluid pressure of 35 pounds per cubic foot (pcf) may be used for level backfill. Where the surface of the retained earth slopes up away from the shoring, a greater pressure should be used. Design data can be developed for additional cases when the design conditions are established.

In addition to the recommended earth pressure, any surcharge (live, including traffic, or dead load) located within a 1:1 plane drawn upward from the base of the shored excavation should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind the wall may be calculated by multiplying the surcharge by 0.5 for the level backfill condition. Lateral load contributions of surcharges located at a distance behind the shored wall may be provided once the load configurations and layouts are known. As a minimum, a 2-foot equivalent soil surcharge (250 psf) is recommended to account for nominal construction loads. It should be noted that the above pressures do not include hydrostatic pressure and assume groundwater will not be encountered in the excavation, or dewatering will be used to lower the ground water table below the bottom of the excavation.

Design of Soldier Piles

All soldier piles should extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. We recommend the required embedment depths be calculated based on the principles of force and moment equilibrium. For this method, the allowable passive pressure against soldier piles that extend below the level of excavation may be assumed to be equivalent to a fluid pressure of 250 pcf. The maximum lateral resistance value should not exceed 3,000 psf. To account for arching, the passive resistance may be assumed to act over a width 3.0 times the width of the embedded portion of the pile, provided adjacent piles are spaced at least 2.5 pile diameters, center-to-center.

Drilling of the soldier pile shafts can be accomplished using heavy-duty drilling equipment. Temporary steel casing may be required to stabilize the sides of the pile shaft. Concrete for piles should be placed immediately after the drilling of the hole is complete. The concrete should be pumped to the bottom of the drilled shaft using a tremie. Once concrete pumping is initiated, a minimum head of 5 feet of concrete above the bottom of the tremie should be established and maintained throughout the concrete placement to prevent contamination of the concrete by soil inclusions. If steel casing is used, the casing should be removed as the concrete is placed.

To develop full lateral resistance, provisions should be taken to assure firm contact between the soldier piles and undisturbed materials. The concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile that is below the planned excavated level should provide sufficient strength to adequately transfer the imposed loads to the surrounding materials.

Lagging

Continuous treated timber lagging should be used between the soldier piles. The lagging should be installed as the excavation proceeds. If treated timber is used, the lagging may remain in place after backfilling. The lagging should be designed for the recommended earth pressure but limited to a maximum value of 400 psf.

Deflection

Shoring adjacent to existing structures or improvements should be designed and constructed to reduce potential movement. The shoring system designer should evaluate potential deflections in their design.

Monitoring

Some deflection of the shored excavation should be anticipated during the planned excavation. We recommend the project civil engineer perform a survey of all existing utilities and structures adjacent to the shored excavation. The purpose of this survey would be to evaluate the ability of existing utility lines or improvements to withstand horizontal movements associated with a shored excavation and to establish the baseline condition in case of unfounded claims of damage. If existing improvements are not capable of withstanding anticipated lateral movements, alternative shoring systems may be required.

Horizontal and vertical movements of the shoring system should be monitored by a licensed surveyor. The construction monitoring and performance of the shoring system are ultimately the contractor's responsibility. However, at a minimum, we recommend that the top of shoring be surveyed prior to excavation and that the top and bottom of the soldier beams be surveyed on a weekly basis until the shoring is not needed. Surveying should consist of measuring movements in vertical and two perpendicular horizontal directions.

4.4 FOUNDATIONS

4.4.1 General

Based on the results of our field exploration, laboratory testing and geotechnical analyses, the proposed retaining walls or culvert (if needed) may be supported on conventional spread foundations placed entirely on engineered fill or competent bedrock. If founded on engineered fill, spread foundations should be underlain by a minimum 2 feet of engineered fill constructed as recommended above. Recommendations for the design lateral earth pressures and design of spread foundations are presented below. Transitions from bedrock to engineered fill beneath a single footing should be avoided. If this condition exists, the bedrock portion should be overexcavated to provide the minimum fill thickness recommended above.

The recommended lateral earth pressures assume that drainage is provided behind the walls to prevent the buildup of hydrostatic pressures. Walls should be provided with drains to reduce the potential for the buildup of hydrostatic pressure. Drains may consist of a 2-foot-wide zone of ¾-inch rock wrapped in filter fabric located immediately behind the wall extending to within 1 foot of the ground surface. Perforated Schedule 40 PVC pipe should be installed within the rock at the base of the drain and sloped to discharge to a suitable collection facility. Commercially available drainage panels could be used as an alternative. The product manufacturer's recommendations should be followed in the installation of a drainage panel. Expansive soils should not be used as wall backfill material.

Where slope extend at inclinations greater than horizontal behind retaining walls, a minimum of a 2-foot width drainage swale should be constructed at the top of the wall to limit the amount of surface water infiltrating behind the wall

4.4.2 Shallow Foundations

Shallow foundation constructed on engineered fill, or entirely on competent bedrock, may be designed for a net allowable bearing pressure of 2,500 pounds per square foot (psf) for dead plus sustained live loads. The foundations should be established at a depth of at least 18 inches below the lowest adjacent exterior grade if founded on soils or at least 12 inches if founded into competent bedrock. A one-third increase in the above bearing pressures can be used for wind or seismic loads.

The structural engineer should design the footing dimension and reinforcement; however shallow foundations should have a minimum width 24 inches. Structurally continuous foundations should not be directly founded on both engineered fill and bedrock. If the proposed foundations are anticipated to directly bear on both engineered fill and bedrock, a structural break should be constructed in the foundation to limit the distress caused by differential settlement. Compaction requirements should follow section 4.3.5 Engineered Fill.

4.4.3 Estimated Settlements

We estimate total static settlement for foundations designed in accordance with the recommendations presented above and supported entirely on engineered fill or bedrock to be less than 1 inch.

4.4.4 Lateral Resistance

Lateral load resistance may be derived from passive resistance along the vertical sides of the foundations, friction acting at the base of the foundation, or a combination of the two. An allowable passive resistance of 250 psf per foot of depth may be used for design. Allowable passive resistance values should not exceed 2,500 psf. An allowable coefficient of friction value of 0.35 between the base of the foundations and the engineered fill soils and competent bedrock can be used for sliding resistance using the dead load forces. An allowable coefficient of friction value of 0.35 between the base of the level concrete pavement and the aggregate base can also be used for sliding resistance using the dead load forces. The pavement sliding friction should be reduced for sloping pavements based on the percentage slope. Friction and passive resistance may be combined without reduction. We recommend that the first foot of soil cover be neglected in the passive resistance calculations.

4.4.5 Lateral Earth Pressures

Design earth pressures for retaining walls depend primarily on the allowable wall movement, wall inclination, type of backfill materials, backfill slopes, surcharges, and drainage. The earth pressures provided assume that that a non-expansive backfill will be used and a drainage system will be installed behind the walls, so that external water pressure will not develop. If a drainage system will not be installed, the wall should be designed to resist hydrostatic pressure in addition to the earth pressure.

The recommended active lateral earth pressures for horizontal backfills using granular relatively non-expansive soils on walls that are free to rotate at least 0.1 percent of the wall height is 35 pcf. The recommended active lateral earth pressures for wall backfills sloping not steeper than 2:1 using granular relatively non-expansive soils on walls that are free to rotate at least 0.1 percent of the wall height is 70 pcf.

The above lateral earth pressures do not include the effects of surcharges (e.g., traffic, footings), compaction, or truck-induced wall pressures. Any surcharge (live, including traffic, or dead load) located within a 1:1 plane drawn upward from the base of the excavation should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind walls may be calculated by multiplying the surcharge by 0.33 for cantilevered walls. Walls adjacent to areas subject to vehicular traffic should be designed for a

2-foot equivalent soil surcharge (240 psf). Lateral load contributions from other surcharges located behind walls may be provided once the load configurations and layouts are known.

4.5 SLOPE STABILITY

In order to reach the grades presented in the MWD Civil Drawings, bedrock cut slopes and engineered fill slopes are designed to be constructed. The proposed bedrock cut slopes up to approximately 25 feet are designed to be excavated to an inclination of 1.5:1 Horizontal:Vertical (H:V) and the fill slopes are designed at 2:1 H:V. We have performed preliminary analysis of the cut and fill slopes to evaluation the feasibility of the proposed slope inclinations. We recommend reevaluating, as needed, the proposed slopes once final plans are prepared.

4.5.1 Methodology

To evaluate the preliminary cut slopes, Kleinfelder completed limit-equilibrium slope stability analyses for the proposed cut slopes using the Slide software by RocScience Inc. (2016). Factors of safety (FOS) for the static and seismic screening analysis were established using Spencer's method. For the screening analysis, the horizontal seismic coefficient was developed using the procedure outlined in SP117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California. We performed a deaggragation based on a recurrence interval of 10 percent in 50 years to develop a design peak ground acceleration of 0.54g. Using the earthquake parameters above, the corresponding seismic coefficient (k_{eq}) is 0.18 for 6 inches (15 cm) of slope displacement.

4.5.2 Cut Slope Stability

We performed analysis on the bedrock cut slopes presented in Cross-Section F as shown on Sheet SK-5 of the MWD Civil Drawings dated February 2018 as well as Cross-Section C as shown on Sheet SK-15 of the MWD Civil Drawings dated April 2018. The parameters selected for the cut slope stability analysis are based on results of direct shear laboratory testing. The results of the laboratory testing are presented below in Table 2.

Table 2

Direct Shear Results and Slope Stability Parameters

Sample Number	Friction Angle (degrees)	Cohesion (psf)
B - 1 at 5 feet	30	150
B – 4 at 5 feet	30	250
B – 5 at 5 feet	41	350
Bedrock Strength Used in Stability Analysis	34	250

Based on the analyses completed, the FOS satisfy the City of Los Angeles minimum required FOS of 1.5 and 1.0 for the static and screening analysis, respectively as shown in Table 3.

Table 3

Bedrock Cut-Slope Stability Analysis Results

Maintenance Road	Analysis	Minimum Required FOS	Calculated FOS
Option 1	Static	1.50	1.54
	Screening	1.00	1.13
Option 2	Static	1.50	1.54
	Screening	1.00	1.18

Note: [1]The screening analysis was performed in accordance with SP117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

The direct shear testing presented in Table 2 was performed on Modified California-Type ring samples. Due to the sampling method and the brittle nature of the bedrock, the strength of the bedrock samples recovered is less than undisturbed intact samples. Although we performed the analysis using the parameters above, we also performed research of CGS Seismic Hazard Zone Report 05 to provide Mean Values for the Chatsworth Formation in the Oat Mountain 7.5-Minute Quadrangle. The Chatsworth Formation shear strength values are shown in Table 4 below and are significantly greater than the parameters included in our analysis.

Table 4

CDMG Published Strength Properties

Bedrock Unit	Mean Friction Angle (degrees)	Mean Cohesion (psf)
Chatsworth Formation	39.3	654

* CGS is formerly California Division of Mines and Geology (CDMG), Seismic Hazard Zone Report 05

4.5.3 Fill Slope Stability

The fill slopes are designed to be constructed at a gradient of 2:1 H:V or greater and do not require slope stability analysis per the current grading code. We anticipate that fill slopes constructed using engineered fill comprised of local materials and sloped at a maximum inclination of 2:1 will be stable

4.5.4 Construction of Permanent Fill Slopes

Fill slopes may be inclined up to 2:1 (horizontal:vertical) or flatter. Where the toe of a fill slope terminates on a natural or cut slope, a keyway is required at the toe of the fill slope. In general, fill slope keyways should be a minimum width of 15 feet, with a minimum depth of 3 feet into competent natural material, and should extend a distance equal to the depth of the keyway beyond the toe of the fill. Benching should be cut into the existing slope to bind the fill to the slope (see Figure 4).

Due to the limited height and configuration of the proposed fill slopes within the portion of the project, slope drains are not anticipated to be needed for this portion of the project. However, depending on fill slope construction and actual site conditions encountered in the field, back drains may be required within the compacted fill to prevent the buildup of hydrostatic pressures behind the fill slope. Field conditions, such as observed seepage from bedrock, or the presence of water within the slope may require the use of subdrains to adequately prevent buildup of hydrostatic pressures behind the fill slope. In general, fill slopes with design heights less than 10 feet will likely not require subdrains. Figures 4 presents standard slope drain details for fill slopes. Benches should be step-like in profile, with each bench not less than four feet in height and established in competent material. Compressible or other unsuitable soils should be removed from the slope prior to benching. Competent material is defined as being essentially free of loose soil, heavy fracturing or erosion prone material and is established by the

Geotechnical Consultant during grading. Following completion of the excavation for the keyway, the project Geotechnical Consultant shall observe the keyway prior to backfilling with certified engineered fill.

When constructing fill slopes the contractor shall avoid spillage of loose material down the face of the slope during dumping and rolling conditions. We recommend that the incoming load be dumped behind the face of the slope and bladed into place. We recommend that fill slopes greater than 10 feet in height should be over-built a minimum of 2 feet in thickness and then trimmed back to expose a compacted core, as shown in Figure 4. The over-built thickness may need to be increased to achieve the specified minimum compaction depending on the site conditions and geometry of the slope. For fill slopes less than 10 feet in height, after 4 feet of vertical height has been obtained, the contractor should compact the outer face of the slope by backing the tamping roller over the top of the slope and thoroughly covering the entire slope surface with overlapping passes of the roller. The foregoing should be repeated after the placement of each 4 foot thickness of fill. Fill slope surface should be compacted to a minimum of 90 percent relative compaction per ASTM D1557.

4.5.5 Construction of Permanent Cut Slopes

In general, cut slopes planned should have a maximum inclination of 1.5:1 (horizontal:vertical). We recommend that a qualified geologist be on site during grading of the cut slopes to map the exposed geology for consistency with the conditions presented in this report. If out-of-slope conditions or other geologic conditions differ from that anticipated then additional analysis and recommendations may be required including trimming the slope to the angle of bedding where practical. If site conditions do not allow trimming the slope to a flatter angle then the slope may need to be over-excavated and replaced with a buttress fill.

4.6 PAVEMENT SECTIONS

4.6.1 Asphalt-Concrete Pavement Sections

The required pavement structural sections will depend on the expected wheel loads, volume of traffic, and subgrade soils. The Traffic Indexes (TI's) assumed should be reviewed by the project Owner, Architect, and/or Civil Engineer to evaluate their suitability for this project. Changes in the TI's will affect the corresponding pavement section. The pavement subgrade should be prepared just prior to placement of the base course. Positive drainage of the paved areas should be provided since moisture infiltration into the subgrade may decrease the life of pavements. The recommended asphalt pavement concrete recommendations are presented below in Table 5.

Table 5
Asphalt Concrete Pavement Sections
(Design R-value = 24)

Traffic Use	Assumed Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
General Roadway Minimum	--	3.0	4.0
Light Access Roadway Traffic	5.0	3.0	7.0

-- denotes minimum pavement thicknesses for flexible pavement design.

The R-value test result evaluated above was 24. We anticipate the final subgrade soils will consist of a blend of the upper and lower fill materials. Since the characteristics of the near-surface soils can change as a result of grading, we recommend that the subgrade soils be retested for pavement support characteristics, to confirm the parameters used in design and allow for a possible reduction in structural section thickness. Pavement sections provided above are contingent on the following recommendations being implemented during construction.

- The pavement sections recommended above should be placed on at least 24 inches of engineered fill compacted to at least 90 percent of maximum dry density with the upper 6 inches compacted to at least 95 percent relative compaction. The overexcavation of the pavement areas should be conducted as recommended in the earthwork section of this report. Prior to fill placement, the exposed subgrade should be scarified to a depth of 8 inches, uniformly moisture conditioned to near optimum moisture content.
- Subgrade soils should be in a stable, non-pumping condition at the time aggregate base materials are placed and compacted.
- Aggregate base materials should be compacted to at least 95 percent relative compaction.
- Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become wet.
- Aggregate base materials should meet current Caltrans specifications for Class 2 aggregate baserock (Class 2), or crushed miscellaneous base (CMB) as specified in "Standard Specifications for Public Work Construction" ("Greenbook").

- The asphalt pavement should be placed in accordance with “Green Book” specifications.
- All concrete curbs separating pavement and landscaped areas should extend into the subgrade and below the bottom of adjacent, aggregate base materials.

Pavement sections provided above are based on the soil conditions encountered during our field investigation, our understanding of the final site grades, and limited laboratory testing. Since the actual pavement subgrade materials exposed during grading may be significantly different than those tested for this study, we recommend that representative subgrade samples be obtained and additional R-value tests performed. Should the results of these tests indicate a significant difference, the design pavement section(s) provided above may need to be revised.

4.6.2 Portland Cement Concrete Pavement

Concrete pavements may be desirable along the alignment. The concrete pavement should have a minimum 28-day compressive strength of 3,000 psi or 4,000 psi as presented below. Control joints should be spaced at every 15 feet or as designed by the Civil Engineer. The concrete pavement section should be placed on at least 24 inches of engineered fill compacted to at least 90 percent of the maximum dry density. Prior to fill placement, the exposed subgrade should be prepared as recommended in Section 4.4 of this report. Table 6 below presents our recommendations of Portland Cement Concrete (PCC) pavement sections.

Table 6

Preliminary Recommended PCC Pavement Sections

Design R-value	Assumed Traffic Index	Concrete Thickness (inches; using a 28-day compressive strength of 3,000 psi)	Concrete Thickness (inches; using a 28-day compressive strength of 4,000 psi)
24	5	7.5	7.0

The PCC sections presented above may be decreased by 0.5 inches provided that they are constructed on 4 inches of Class 2 aggregate base or CMB compacted to 95% relative compaction. We recommend that the additional 4 inches of aggregate base described above should also underlain 24 inches of engineered fill compacted to at least 90 percent relative compaction. Our review of the MWD Civil Drawings presents details including a 9-inch PCC thickness, which is also acceptable for our understanding of the traffic loading conditions.

5 RECOMMENDED ADDITIONAL SERVICES

5.1 ADDITIONAL GEOTECHNICAL INVESTIGATION

Our authorized scope included limited geotechnical investigation. Conditions could vary between the locations explored. We do not anticipate encountering adverse bedding conditions during grading. However, if adverse bedding conditions are encountered, redesign of proposed slopes may be necessary resulting in delays during construction. To reduce the risk of construction delays, confirmation borings could be excavated through the top of each proposed cut slope prior to construction. Kleinfelder can provide a proposal for additional scope and fee if this option is desired. A geotechnical representative should be retained to provide full-time observation and geologic mapping during construction of all slopes constructed for this project.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that Kleinfelder perform a general review of the project plans and specifications before they are finalized to verify that our geotechnical recommendations have been properly interpreted and implemented during design. If we are not accorded the privilege of performing this review, we can assume no responsibility for misinterpretation of our recommendations.

5.3 CONSTRUCTION OBSERVATION AND TESTING

The construction process is an integral design component with respect to the geotechnical aspects of a project. Because geotechnical engineering is an inexact science due to the variability of natural processes, and because we sample only a limited portion of the soils affecting the performance of the proposed structure, unanticipated or changed conditions can be encountered during grading. Proper geotechnical observation and testing during construction are imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process. Therefore, we recommend that Kleinfelder be retained during the construction of the proposed improvements to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study.

Our services are typically needed at the following stages of grading.

- after demolition;
- during grading;
- after the overexcavation, but prior to scarification;
- during utility trench backfill;
- during base placement and site paving; and
- after excavation for foundations.

6 LIMITATIONS

This geotechnical study has been prepared for the exclusive use by Metropolitan Water District (Client) and their agents for specific application to the project in Chatsworth, California. The findings, conclusions and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No other warranty, express or implied, is made.

The scope of services was limited to a background data review and the field exploration described in the Scope of Services section. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on our field exploration and laboratory testing programs, and engineering analyses.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the locations of the improvements, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid until the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, preparation of foundations, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the engineer of record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than one year from the date of the report. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party, other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of this report and the nature of the new project, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party and the client agrees to defend, indemnify, and hold harmless Kleinfelder from any claims or liability associated with such unauthorized use or non-compliance.

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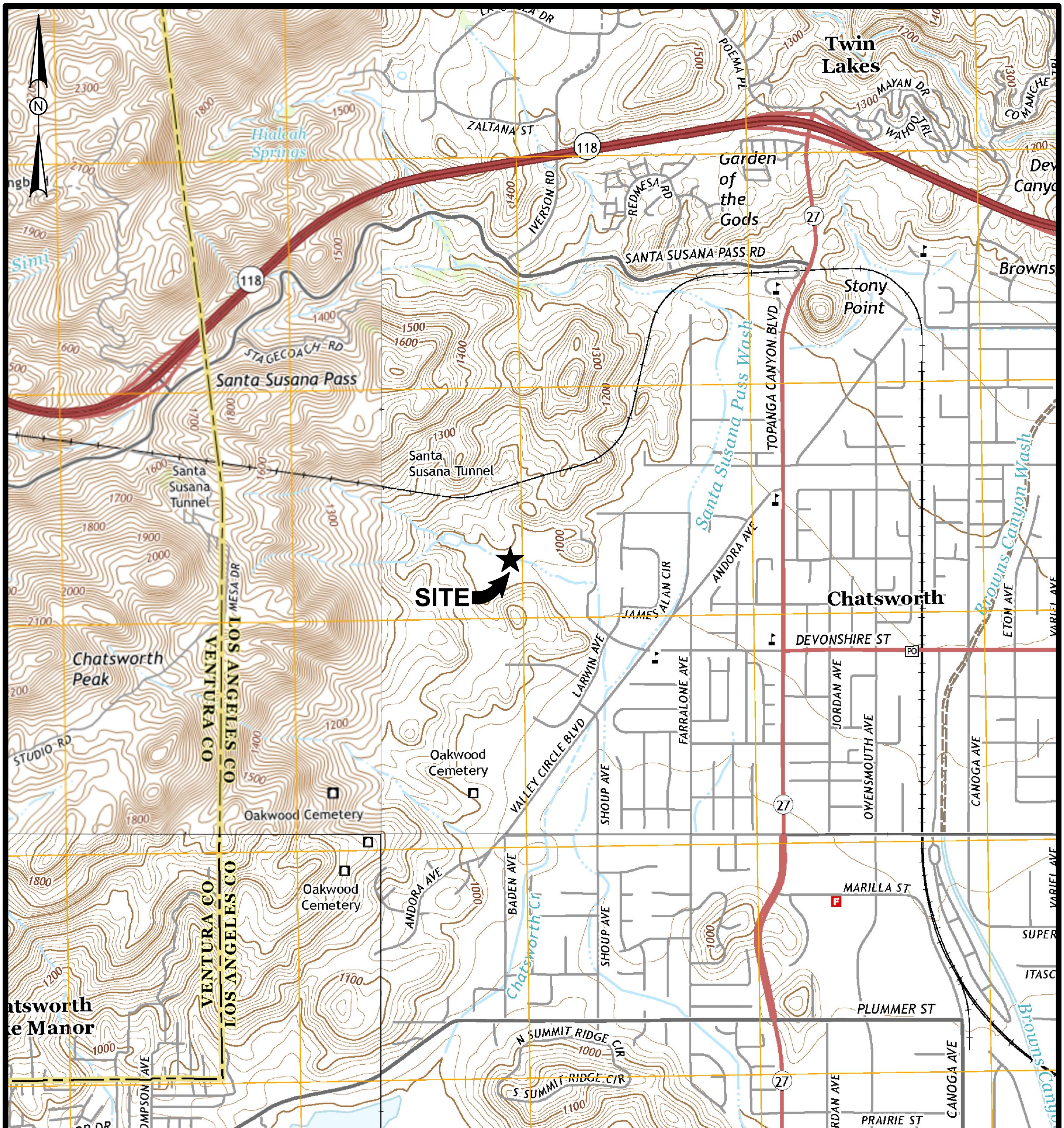
FIGURES

PLOTTED: 2/16/2018 1:39 PM BY: dean fahrney



SOURCE: U.S.G.S. 7.5' Topographic series, Oat Mountain, California Quadrangle 2015.

2,000 1,000 0 2,000

APPROXIMATE SCALE (feet)

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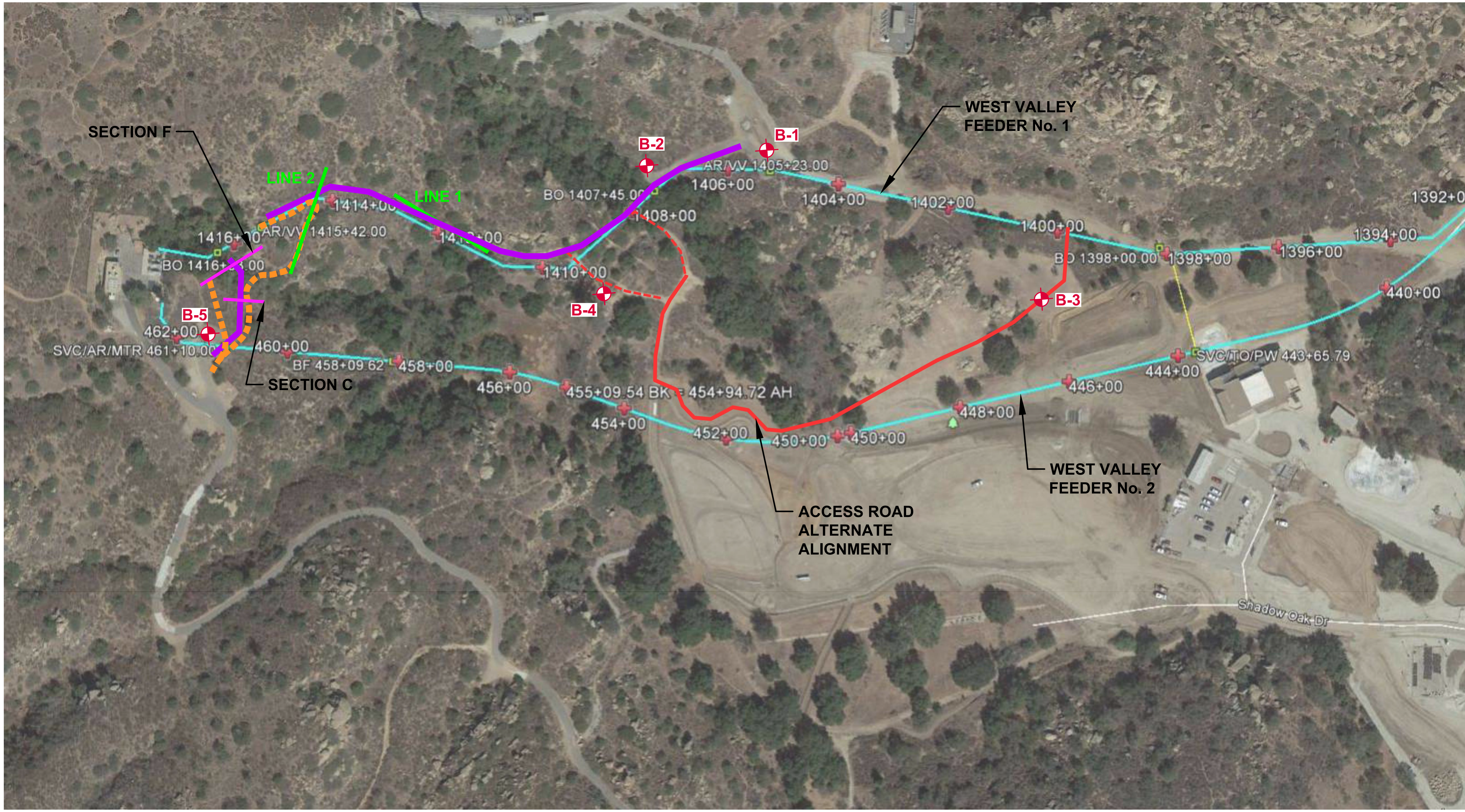
PROJECT NO.	20180213	SITE VICINITY MAP	FIGURE
DRAWN BY:	DMF		
CHECKED BY:	JDW	WEST VALLEY FEEDER 1 VALVE IMPROVEMENTS CHATSWORTH, CALIFORNIA	1
DATE:	02/2018		
REVISED:	02/2018		

CAD FILE: U:\Projects\CADD\CADD 2018\20180213\West-Valley-Feeder\20180213_F1.dwg



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200 100 0 200

APPROXIMATE SCALE (feet)

EXPLANATION

B-5 APPROXIMATE BORING LOCATION

LINE 2 APPROXIMATE SEISMIC REFRACTION LINE LOCATION

APPROXIMATE LOCATION OF ACCESS ROAD ALIGNMENT OPTION 1

APPROXIMATE LOCATION OF ACCESS ROAD ALIGNMENT OPTION 2

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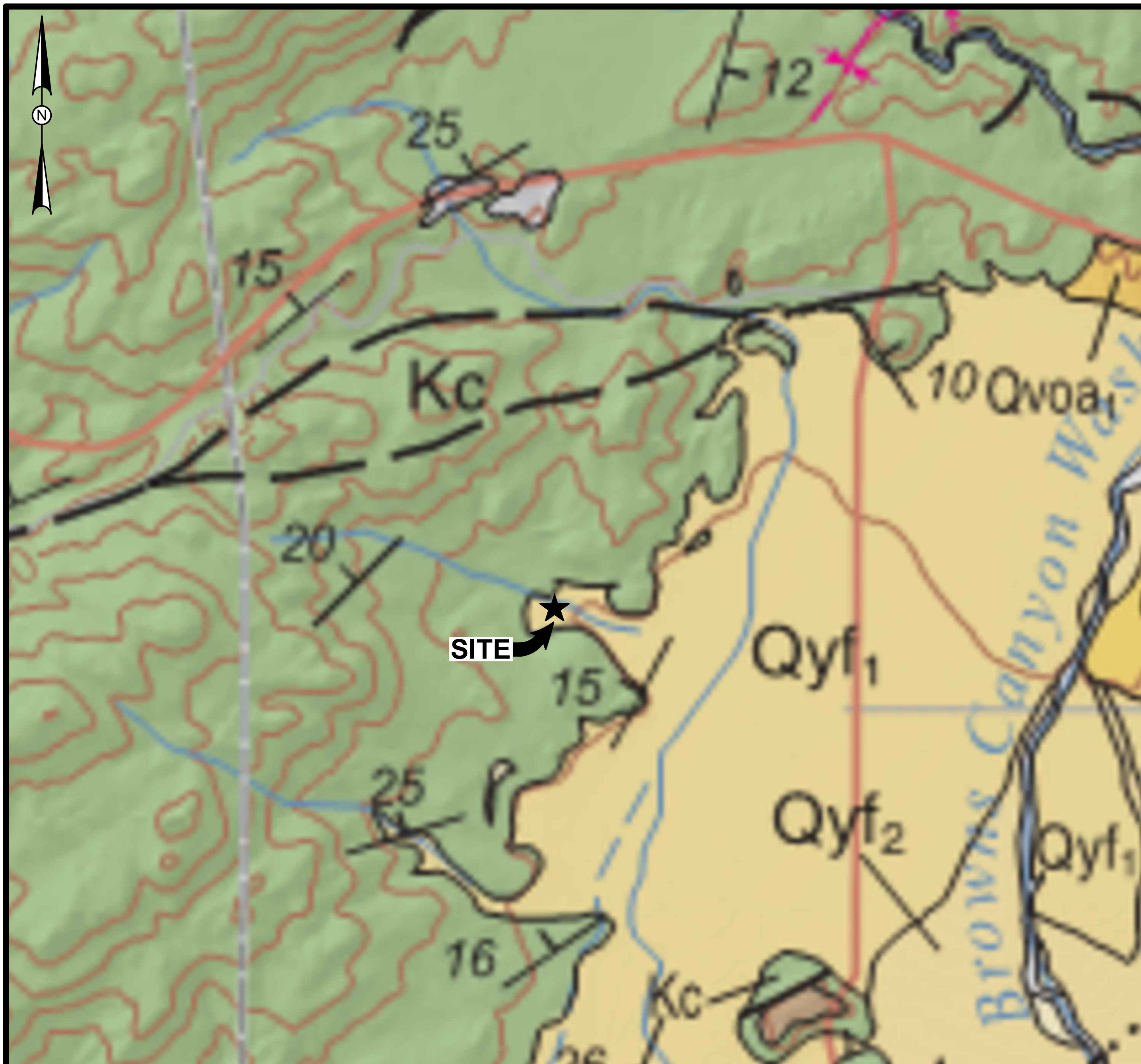
PROJECT NO. 20180213	FIELD EXPLORATION LOCATION MAP	FIGURE
DRAWN BY DMF		2
CHECKED BY JDW	WEST VALLEY FEEDER 1 VALVE IMPROVEMENTS CHATSWORTH, CALIFORNIA	
DATE: 02/2018		
REVISED: 02/2018		

PLOTTED: 5/11/2018 9:42 AM BY: dean fahrney
LAYOUT: 1
CAD FILE: U:\Projects\CADD\CADD 2018\20180213\West-Valley-Feeder\20180213_F2.dwg



SOURCE: PRELIMINARY GEOLOGIC MAP OF THE LOS ANGELES 30'x60' QUADRANGLE, CALIFORNIA VERSION 2.1 COMPILED BY RUSSELL H. CAMPBELL, CHRIS J. WILLIS, PAMELA J. IRVINE, AND BRIAN J. SWANSON, 2014

2,000 1,000 0 2,000
APPROXIMATE SCALE (feet)

EXPLANATION

Qyf_2	YOUNG ALLUVIAL FAN DEPOSITS, UNIT 2
Qyf_1	YOUNG ALLUVIAL FAN DEPOSITS, UNIT 1
Qvoa	VERY OLD ALLUVIUM, UNDIVIDED
Trn	RINCON FORMATION, MARINE SHALE AND MUDSTONE
Kc	CHATSWORTH FORMATION

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.	20180213
DRAWN BY:	DMF
CHECKED BY:	JDW
DATE:	02/2018
REVISED:	02/2018

REGIONAL GEOLOGIC MAP

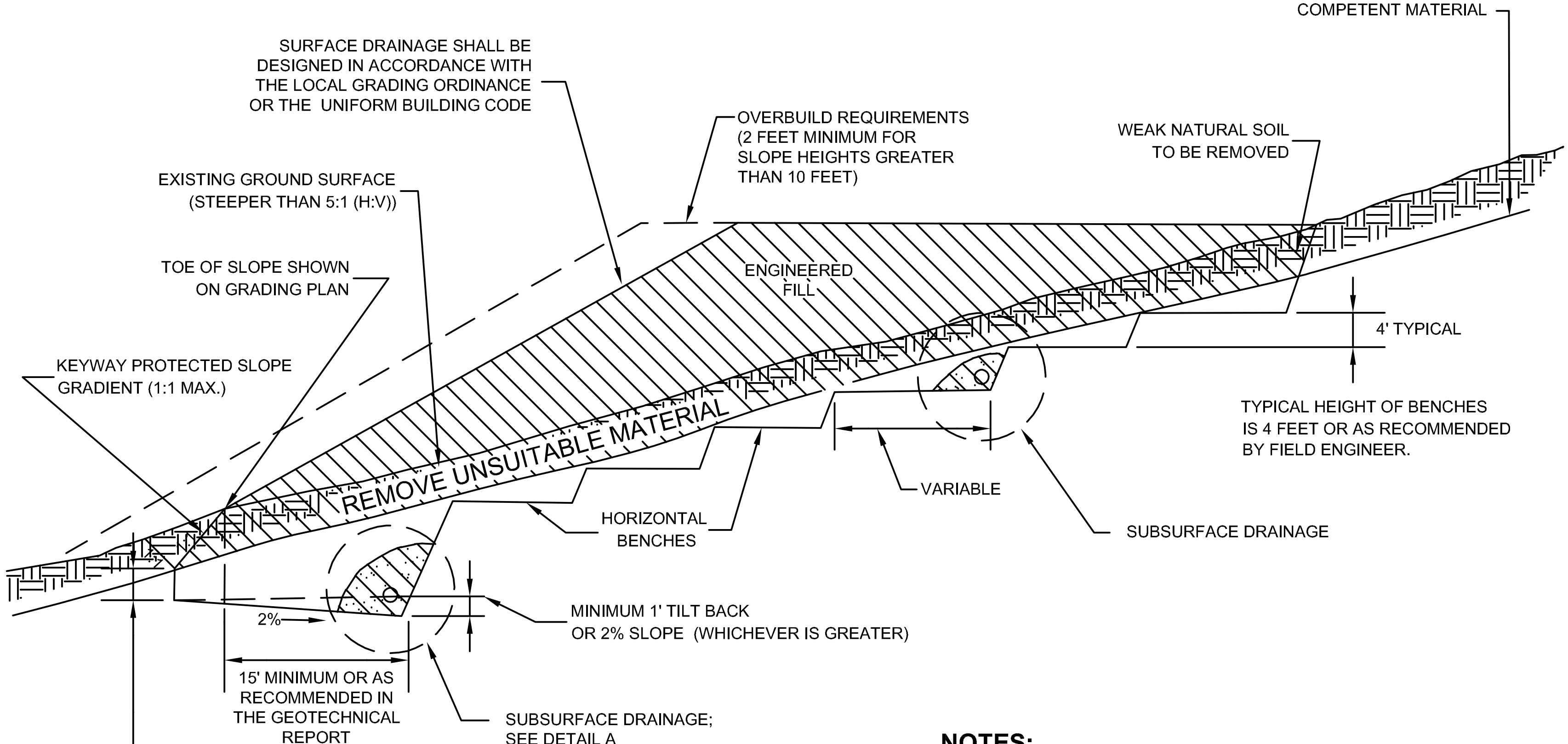
WEST VALLEY FEEDER 1 VALVE IMPROVEMENTS
CHATSWORTH, CALIFORNIA

FIGURE
3

ATTACHED IMAGES:
ATTACHED XREFS:
RIVERSIDE, CA

CAD FILE: U:\Projects\CADD\CADD 2018\20180213\West-Valley-Feeder\20180213_F4.dwg

PLOTTED: 23 Mar 2018, 4:06pm, KHagan

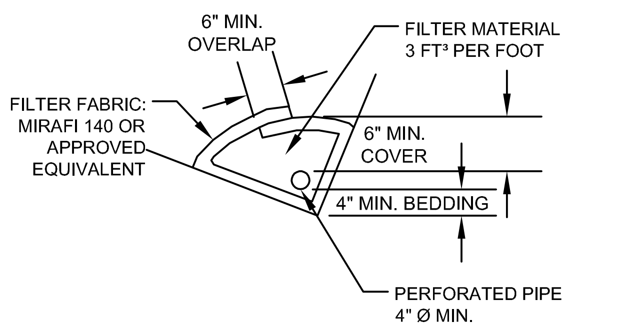


NOTES:

1. FINAL BENCHING AND KEYWAY EXCAVATION DEPTHS AND DETAILS SUBJECT TO EVALUATION OF FIELD CONDITIONS.
2. DIMENSIONS AND LOCATIONS OF SUB DRAINAGE ARE APPROXIMATE. ACTUAL INSTALLATION TO BE BASED ON FIELD CONDITIONS.
3. GRADE PIPES TO DRAIN TO AN APPROPRIATE DISCHARGE FACILITY.
4. INSTALL CLEANOUTS AT ENDS, ANGLES, AND JUNCTIONS OF PIPE.
5. BENCH SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE SOIL ENGINEER.
6. SUBDRAINAGE: BACK DRAIN REQUIRED AS RECOMMENDED IN THE GEOTECHNICAL REPORT AND VERIFIED IN THE FIELD BY THE GEOTECHNICAL CONSULTANT

DETAIL A:

PERFORATED PIPE SURROUNDED BY FILTER MATERIAL



FILTER MATERIAL:
3/4" Gravel or an approved alternate.

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TYPICAL ENGINEERED FILL SLOPE/BENCHING DETAIL

WEST VALLEY FEEDER 1 VALVE IMPROVEMENTS
CHATSWORTH, CALIFORNIA

FIGURE
4

APPENDIX A
Field Explorations

APPENDIX A

FIELD EXPLORATIONS

The subsurface exploration program for the proposed project consisted of advancing and logging a total of 5 hollow-stem auger borings. The borings were drilled with a limited access track drill rig equipped with 8-inch diameter hollow-stem augers, provided by 2R Drilling of Chino, California. The approximate locations of the borings are shown on Figure 2, Field Exploration Location Map.

The logs of the borings are presented on Figures A-3 through A-7. An explanation to the logs is presented on Figures A-1 and A-2, Soil Description Key and Graphics Key, respectively. The logs of borings present a description of the earth materials encountered, samples obtained, and show field and laboratory tests performed. The logs also show the boring number, drilling date, boring elevation and the name of the logger and drilling subcontractor. A Kleinfelder staff professional logged the borings utilizing the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers and/or bedrock may be gradual. Bulk and drive samples of representative earth materials were obtained from the borings at maximum intervals of approximately 5 feet. With the exception of Boring B-3, the excavated soil cuttings were used to backfill the excavations. In Boring B-3, the boring was backfilled with cement/bentonite grout.

A California sampler was used to obtain relatively undisturbed drive samples of the soil encountered. This sampler consists of a 3 inch O.D., 2.5 inch I.D. split barrel shaft that is driven a total of 18 inches into the soil at the bottom of the boring. The soil was retained in six 1-inch brass rings for laboratory testing. The sampler was driven using a 140-pound automatic hammer falling 30 inches. The total number of hammer blows required to drive the sampler the final 12 inches is termed the blow count and is recorded on the Logs of Borings. Where the sample was driven less than 12 inches, the number of blows to drive the sample for each 6-inch segment, or portion thereof, is shown on the logs.

Bulk samples of the sub-surface soils were directly retrieved from the soil cuttings produced by the auger blades.

SAMPLER AND DRILLING METHOD GRAPHICS

- BULK / GRAB / BAG SAMPLE
- MODIFIED CALIFORNIA SAMPLER (2 or 2-1/2 in. (50.8 or 63.5 mm.) outer diameter)
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)
- HQ CORE SAMPLE (2.500 in. (63.5 mm.) core diameter)
- SHELBY TUBE SAMPLER
- HOLLOW STEM AUGER
- SOLID STEM AUGER
- WASH BORING
- SONIC CONTINUOUS SAMPLER

GROUND WATER GRAPHICS

- WATER LEVEL (level where first observed)
- WATER LEVEL (level after exploration completion)
- WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

ABBREVIATIONS

WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

Major Division	Subdivision	Fines	Criteria	Symbol	Description
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	**GRAVELS** (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu≥4 and 1≤Cc≤3	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu<4 and/ or 1>Cc>3	**GP**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	Cu≥4 and 1≤Cc≤3	**GW-GM**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu<4 and/ or 1>Cc>3	**GP-GM**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		**GM**	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu≥6 and 1≤Cc≤3	**SW**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu<6 and/ or 1>Cc>3	**SP**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	Cu≥6 and 1≤Cc≤3	**SW-SM**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu<6 and/ or 1>Cc>3	**SP-SM**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		**SM**	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

Major Division	Subdivision	Symbol	Description
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	**ML**	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

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PROJECT NO.:	20174481
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CHECKED BY:	JW
DATE:	3/1/2018
REVISED:	-

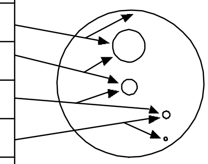
GRAPHICS KEY

West Valley Feeder 1 Valve Improvements
Chatsworth, CA

FIGURE
A-1

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



SECONDARY CONSTITUENT

Term of Use	AMOUNT: Secondary Constituent is Fine Grained	AMOUNT: Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
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

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N_{60} (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q_u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25≤ PP <0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5≤ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1≤ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2≤ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4≤ PP	>8000	Thumbnail will not indent soil.

FROM TERZAGHI AND PECK, 1948; LAMBE AND WHITMAN, 1969; FHWA, 2002; AND ASTM D2488

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

FROM TERZAGHI AND PECK, 1948

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.
High (H)	> 50	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 or thread can be formed without crumbling when drier than the plastic limit.

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness.
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

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PROJECT NO.: 20174481	SOIL DESCRIPTION KEY	FIGURE
DRAWN BY: CC		
CHECKED BY: JW	West Valley Feeder 1 Valve Improvements Chatsworth, CA	A-2
DATE: 3/1/2018		
REVISED: -		

BORING LOG B-1

Date Begin - End:	1/30/2018	**Drilling Company:**	2R Drilling
Logged By:	C.Coffey	**Drill Crew:**	Jeff
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-75 Track Mounted
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	Party Cloudy/Mid 80s	**Exploration Diameter:**	8 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

Approximate Ground Surface Elevation (ft.): 1,034.00
Surface Condition: Gravel

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description (FIELD EXPLORATION)	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%) (LABORATORY RESULTS)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	0		**Fill** **Silty SAND (SM)**: fine-grained, pale olive brown, dry, some medium sand, subangular											hand auger to 0.5 feet, refusal at 0.5 feet due to very dense soil/bedrock
	2		**Bedrock: Chatsworth Fm. (Kc)** **SANDSTONE**: fine to medium-grained, yellowish brown (10YR-3/6), dry to moist, very dense, excavates as silty sand (SM) with gravel with fine to medium gravel (0.5 to .75 inch)		BC=50/2"			3.6	104.3					
1030														
	5				BC=50/5"									direct shear test
1025			excavates as silty sand (SM), fine grained, some fine gravel, subangular, slightly friable											hard drilling
	10				BC=50/1"			4.4	104.4					

The boring was terminated at approximately 11.5 ft. below ground surface. The boring was backfilled with soil on January 30, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20174481 DRAWN BY: CC CHECKED BY: JW DATE: 3/1/2018 REVISED: -	BORING LOG B-1 West Valley Feeder 1 Valve Improvements Chatsworth, CA	FIGURE A-3

BORING LOG B-2

Date Begin - End:	1/30/2018	**Drilling Company:**	2R Drilling
Logged By:	C.Coffey	**Drill Crew:**	Jeff
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-75 Track Mounted
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	Party Cloudy/Mid 80s	**Exploration Diameter:**	8 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

FIELD EXPLORATION / LABORATORY RESULTS

Approximate Ground Surface Elevation (ft.): 1,018.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	0	Asphalt: 3-inches					7.0						Hand auger to 5 feet
		Base: 8-inches											
		Alluvium **Silty SAND (SM)**: fine-grained, dark yellowish brown (10YR-3/8), dry to moist, some medium sand, subangular to angular											corrosion test
1015													
		Bedrock:Chatsworth Fm. (Kc) **SANDSTONE**: fine-grained, yellowish brown (10YR 3/4) to very dark brown (10YR-2/2), moist, very dense, excavates as well-graded sand with silt (SW-SM)											
	5			BC=7 50/5"		SP-SM	12.2	114.2	100	11			
1010													hard drilling
		yellowish brown (10YR-3/4), excavates as silty sand (SM)											
	10			BC=50/3"									
1005													
		fine to medium-grained, light brownish gray (10YR-6/4), excavates as poorly graded sand with Gravel (SP), fine to medium (up to 1.5-inch) angular gravel											
	15			BC=50/2"									
1000													
	20												
995													

The boring was terminated at approximately 16.5 ft. below ground surface. The boring was backfilled with soil on January 30, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

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PROJECT NO.: 20174481	BORING LOG B-2		FIGURE
DRAWN BY: CC			A-4
CHECKED BY: JW	West Valley Feeder 1 Valve Improvements Chatsworth, CA		
DATE: 3/1/2018			
REVISED: -			PAGE: 1 of 1

BORING LOG B-3

Date Begin - End:	1/30/2018	**Drilling Company:**	2R Drilling
Logged By:	C.Coffey	**Drill Crew:**	Jeff
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-75 Track Mounted
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	Party Cloudy/Mid 80s	**Exploration Diameter:**	8 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

Approximate Ground Surface Elevation (ft.): 996.00
Surface Condition: Soil

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description (FIELD EXPLORATION)	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol (LABORATORY RESULTS)	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			Alluvium **Silty SAND (SM)**: fine-grained, yellowish brown (10YR-5/6), dry to moist, very dense, weak to medium cemented					6.3						hand auger to 5 feet
995														R-Value test
					BC=30 50/6"			6.7	113.3	100	40			
990	5		becomes yellow (2.5Y-7/6) weakly cemented		BC=22 19 32									
985	10		increase in fines, becomes moist, olive brown (2.5Y-4/3)		BC=12 25 32									
			becomes medium dense											
980	15				BC=10 15 15									
975	20													

The boring was terminated at approximately 16.5 ft. below ground surface. The boring was backfilled with cement/bentonite grout on January 30, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

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PROJECT NO.:	20174481
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DATE:	3/1/2018
REVISED:	-

BORING LOG B-3

West Valley Feeder 1 Valve Improvements
Chatsworth, CA

FIGURE

A-5

Date Begin - End:	1/30/2018	**Drilling Company:**	2R Drilling	**BORING LOG B-4**
Logged By:	C.Coffey	**Drill Crew:**	Jeff	
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-75 Track Mounted	**Hammer Type - Drop:** 140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger	
Weather:	Party Cloudy/Mid 80s	**Exploration Diameter:**	8 in. O.D.	

FIELD EXPLORATION / LABORATORY RESULTS

Approximate Ground Surface Elevation (ft.): 1,021.00
Surface Condition: Soil

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
1020	0–5		**Alluvium** **Sandy Lean CLAY (CL)**: low plasticity, dark yellowish brown (10YR-3/4), dry, fine sand					6.4		100	58			Hand auger to 5 feet, medium to hard hand augering
1015	5		**Bedrock: Chatsworth FM (Kc)** **SANDSTONE**: fine-grained, very pale brown (10YR-7/4), dry, very dense, excavates as silty sand (SM), slightly friable		BC=26 34 40									direct shear test
1010	10				BC=50/5"			6.0	111.2					
1005	15				BC=50/2"									
1000	20													

The boring was terminated at approximately 16.5 ft. below ground surface. The boring was backfilled with soil on January 30, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

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PROJECT NO.: 20174481	BORING LOG B-4		FIGURE
DRAWN BY: CC			
CHECKED BY: JW	West Valley Feeder 1 Valve Improvements Chatsworth, CA		A-6
DATE: 3/1/2018			
REVISED: -			

BORING LOG B-5

Date Begin - End:	1/30/2018	**Drilling Company:**	2R Drilling		
Logged By:	C.Coffey	**Drill Crew:**	Jeff		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-75 Track Mounted	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Party Cloudy/Mid 80s	**Exploration Diameter:**	8 in. O.D.		

FIELD EXPLORATION / LABORATORY RESULTS

Approximate Ground Surface Elevation (ft.): 1,116.00
Surface Condition: Soil

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Blow Counts(BC)= Uncorr. Blows/6 in.	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
1115		**Fill** **Poorly Graded SAND with Silt (SP-SM)**: fine-grained, dark yellowish brown (10YR-4/4), dry, dense, moderately cemented				6.4						hand auger to 0.5 feet, hand auger refusal at 0.5 feet due to dense bedrock/sandstone; compaction test
		Bedrock: Chatsworth Fm. (Kc) **SANDSTONE**: dark yellowish brown (10YR-4/4), very dense, excavates as silty sand with gravel (SM)	BC=16 22 30			6.1	118.3	79	17			R-Value test
1110	5		BC=8 12 50/5"									
1105	10	weakly cemented, increase in fines	BC=50/3"			5.8	98.0					
												hard drilling
1100	15	excavates as silty sand (SM), moderately cemented	BC=50/2"									disturbed sample
1095	20	moderately cemented	BC=50/2"									disturbed sample

The boring was terminated at approximately 21.5 ft. below ground surface. The boring was backfilled with soil on January 30, 2018.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

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PROJECT NO.:	20174481
DRAWN BY:	CC
CHECKED BY:	JW
DATE:	3/1/2018
REVISED:	-

BORING LOG B-5

West Valley Feeder 1 Valve Improvements
Chatsworth, CA

FIGURE
A-7

APPENDIX B

LABORATORY TESTING

GENERAL

Laboratory tests were performed on selected samples as an aid in classifying the soils and to evaluate physical properties of the soils that may affect foundation design and construction procedures. The tests were performed in general conformance with the current ASTM or California Department of Transportation (Caltrans) standards. A description of the laboratory-testing program is presented below.

Laboratory tests were performed on representative relatively undisturbed and bulk soil samples to estimate engineering characteristics of the various earth materials encountered. Testing was performed in accordance with one of the following references:

1. Lambe, T. William, Soil Testing for Engineers, Wiley, New York, 1951.
2. Laboratory Soils Testing, U.S. Army, Office of the Chief of Engineers, Engineering Manual No. 1110-2-1906, November 30, 1970.
3. ASTM Standards for Soil Testing, latest revisions.
4. State of California Department of Transportation, Standard Test Methods, latest revisions.

LABORATORY MOISTURE AND DENSITY DETERMINATIONS

Natural moisture content and dry density tests were performed on selected soil samples collected. Moisture content was evaluated in general accordance with ASTM Test Method D 2216; dry unit weight was evaluated using procedures similar to ASTM Test Method D 2937. The results are presented on the Logs of Borings and are summarized in Table B-1, Moisture Content and Unit Weight.

SIEVE AND HYDROMETER ANALYSIS

Sieve analyses were performed on four samples and Hydrometer Analysis was performed on one sample of the materials encountered at the site to evaluate the grain size distribution characteristics of the soils and to aid in their classification. The tests were performed in general accordance with ASTM Test Method D 422. The test results are presented as Figures B-1 and B-2, Grain Size Distribution Curve.

DIRECT SHEAR

Direct shear testing was conducted on five samples to evaluate the shear strength parameters of representative on-site soils. The samples from B-1 and B-5 was taken from a bulk sample and remolded to 90% relative compaction for the test. Each sample was tested in a saturated state in general accordance with ASTM Test Method D3080-90. The test results are presented on Figure B-3 through B-7, Direct Shear Test.

EXPANSION INDEX

Expansion index testing was performed on a sample of the subsurface soils to evaluate their expansion characteristics. The test was performed in accordance with UBC Standard No. 18-2, Expansion Index Test Method. The test result is presented on Table B-2, Expansion Index Test Result and may be compared to the table presented below to qualitatively evaluate the expansion potential of the near-surface site soils.

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

MAXIMUM DENSITY/OPTIMUM MOISTURE TEST

Four maximum density/optimum moisture tests were performed on select bulk samples of the on-site soils to determine compaction characteristics. The tests were performed in accordance with ASTM Standard Test Method D-1557-91. The test results are presented in Table B-3, Maximum Density / Optimum Moisture Test Results.

R-VALUE TEST

Three resistance value (R-value) tests were performed to evaluate support characteristics of the near-surface onsite soils. R-value testing was performed in accordance with Caltrans Standard Test Method 301. The test results are presented in Table B-4, R-Value Test Results.

PRELIMINARY CORROSIVITY TESTS

A series of chemical tests were performed on two representative soil samples collected from the borings to estimate pH, sulfate content, chloride content, and electrical resistivity. The test results may be used by a qualified corrosion engineer to evaluate the general corrosion potential with respect to the construction materials. The results of the tests are presented in Table B-5, Preliminary Corrosion Test Results.

Table B-1

Moisture Content and Unit Weight

Boring	Depth (ft)	Moisture Content (%)	Dry Unit Weight (pcf)
B-1	2	3.6	104.3
B-1	10	4.4	104.4
B-2	0 – 5	7.0	--
B-2	5	12.2	114.2
B-2	10	10.1	106.3
B-3	0 – 5	6.3	--
B-3	2	6.7	113.3
B-3	5	5.0	104.9
B-4	0 – 5	6.4	--
B-4	10	6.0	111.2
B-5	0 – 5	6.4	--
B-5	2	6.1	118.3
B-5	10	5.8	98.0

– denotes dry unit weight test was not performed due to sample type

Table B-2

Expansion Index Test Result

Boring	Depth (ft)	Expansion Index	Expansion Potential
B-4	0 – 5	56	Medium

Table B-3
Maximum Density/Optimum Moisture Test Results

Boring	Depth (ft)	Maximum Density (pcf)	Optimum Moisture (%)
B – 1	0 – 5	128.6	8.2
B – 5	0 – 5	130.3	8.2

Table B-4

R-Value Test Results

Boring	Approximate Depth (ft)	R-Value
B – 3	0 – 5	19
B – 5	0 – 5	29

Table B-5

Preliminary Corrosion Test Results

Boring	Depth (ft)	pH	Sulfate (ppm)	Chloride (ppm)	Resistivity (ohm-cm)
B – 2	0 – 5	8.3	1981	55	48

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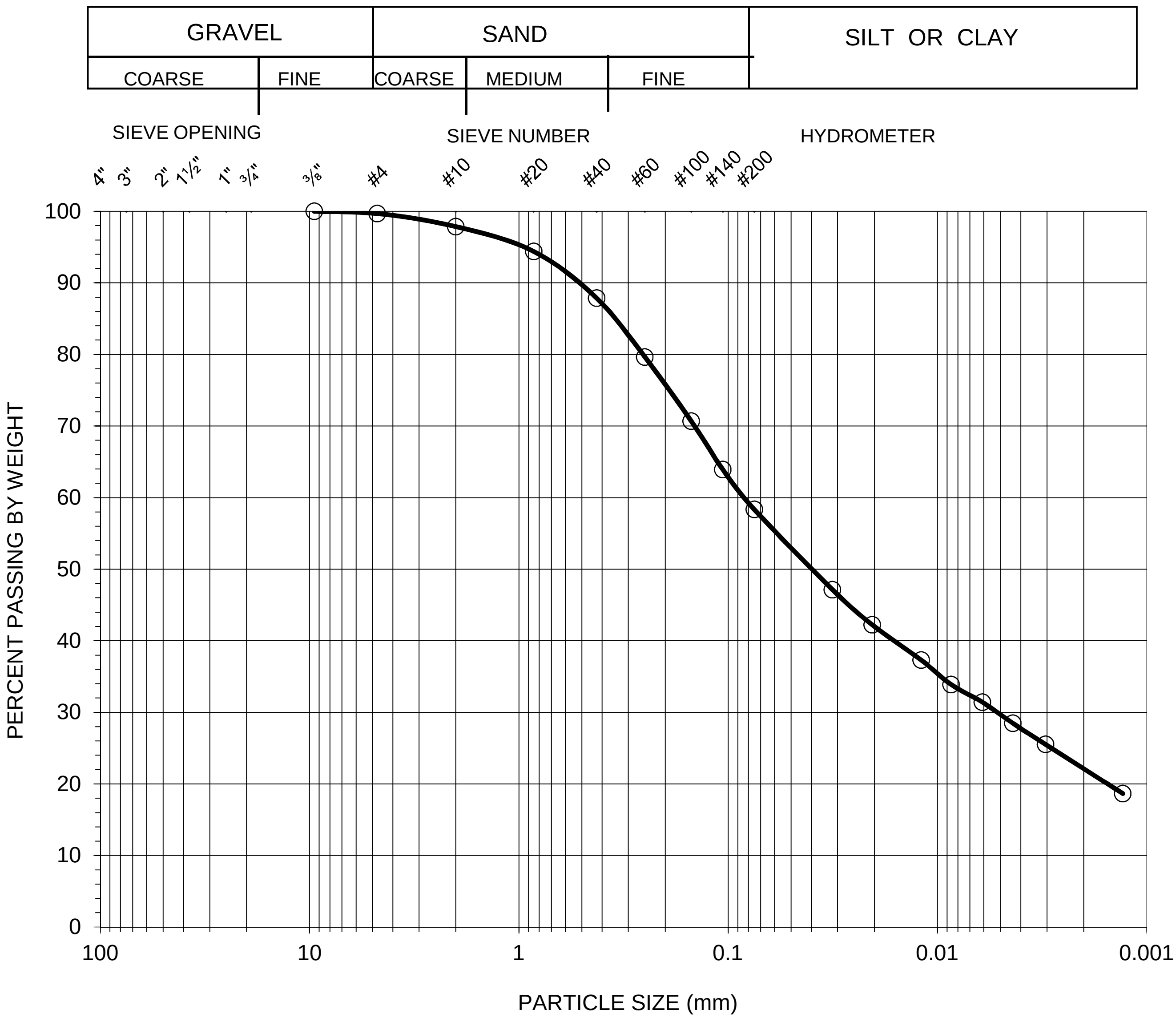
Figure B-1

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913 & D 7928

Client Name:	Kleinfelder	Tested by:	JT	Date:	02/09/18
Project Name:	Municipal Water District - West Valley Feeder	Computed by:	JP	Date:	02/09/18
Project Number:	20180213.002A	Checked by:	AP	Date:	02/09/18



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent Gravel	Percent Sand	Percent Silt & Clay	Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
○	B-4	1	0-5	0	42	58	N/A	CL*

Note: Based on visual classification of sample

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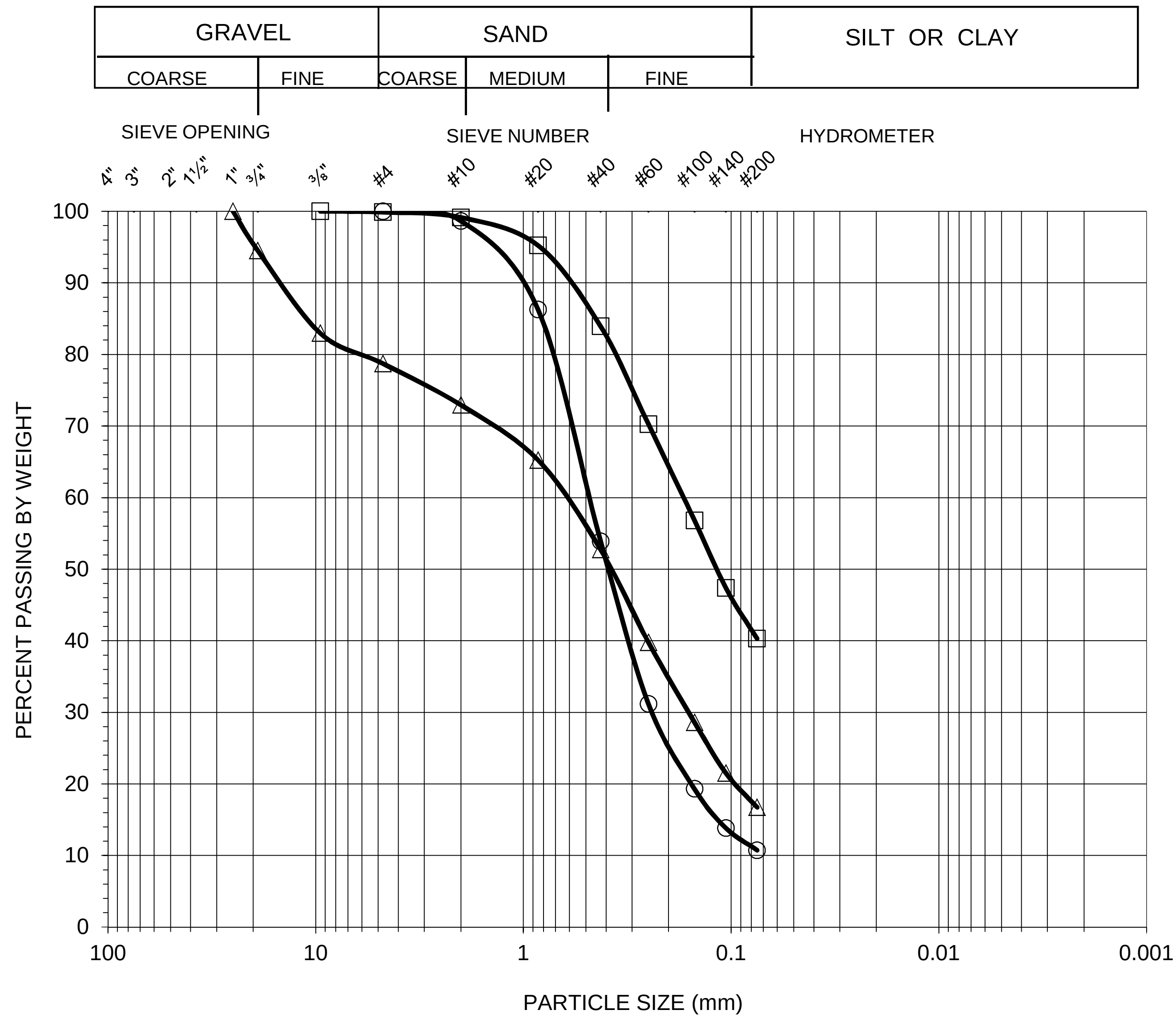
Figure B-2

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	Kleinfelder	Tested by:	JT	Date:	02/09/18
Project Name:	Municipal Water District - West Valley Feeder	Computed by:	JP	Date:	02/09/18
Project Number:	20180213.002A	Checked by:	AP	Date:	02/09/18



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	B-2	2	5	0	89	11	N/A	SW-SM
□	B-3	2	2	0	60	40	N/A	SC*
△	B-5	2	2	21	62	17	N/A	SM

**Note: Based on visual classification of sample*

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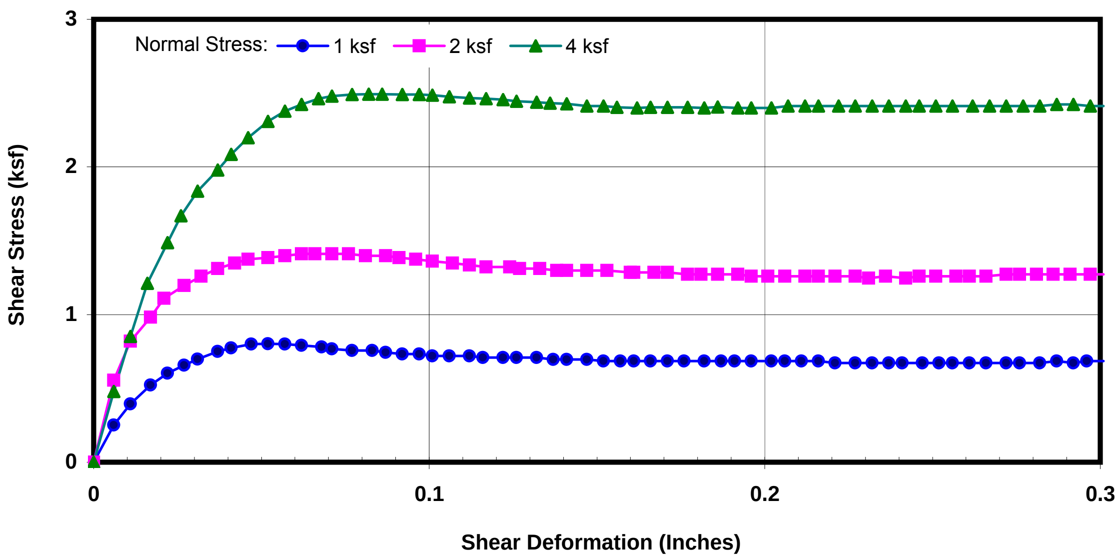
Figure B-3

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	Municipal Water District - West Valley Feeder	**Tested By:** LS	**Date:** 02/26/18
Project No.:	20180213.002A	**Computed By:** JP	**Date:** 02/27/18
Boring No.:	B-1	**Checked by:** AP	**Date:** 02/27/18
Sample No.:	1 **Depth (ft):** 0-5		
Sample Type:	Remolded to 90% RC at opt. MC		
Soil Description:	Silty Sand		
Test Condition:	Inundated **Shear Type:** Regular		

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
125.6	115.7	8.5	15.3	50	90	1	0.802	0.684
						2	1.411	1.273
						4	2.493	2.412



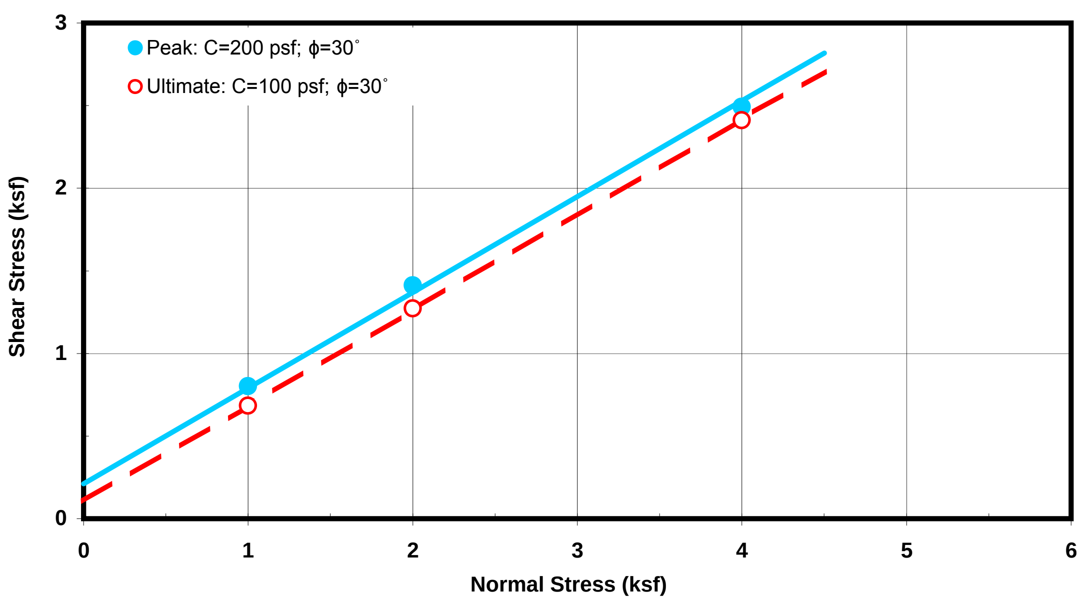


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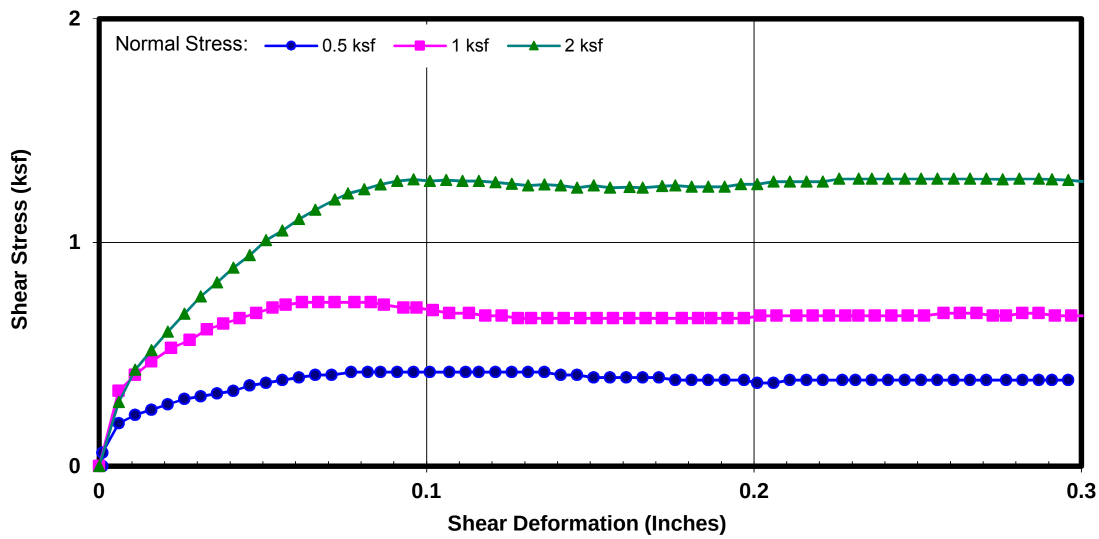
Figure B-4

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Municipal Water District - West Valley Feeder
Project No.: 20180213.002A
Boring No.: B-1
Sample No.: 3 **Depth (ft):** 5
Sample Type: Mod. Cal.
Soil Description: Silty Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 02/06/18
Computed By: JP **Date:** 02/07/18
Checked by: AP **Date:** 02/09/18

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
112.5	107.6	4.6	18.9	22	90	0.5	0.420	0.384
						1	0.732	0.672
						2	1.284	1.272



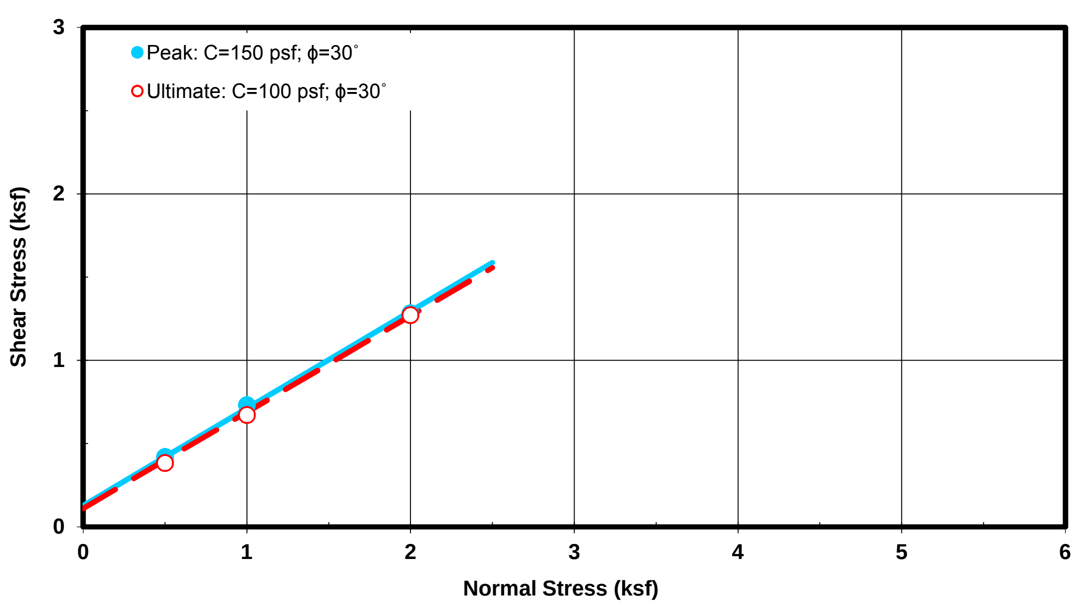


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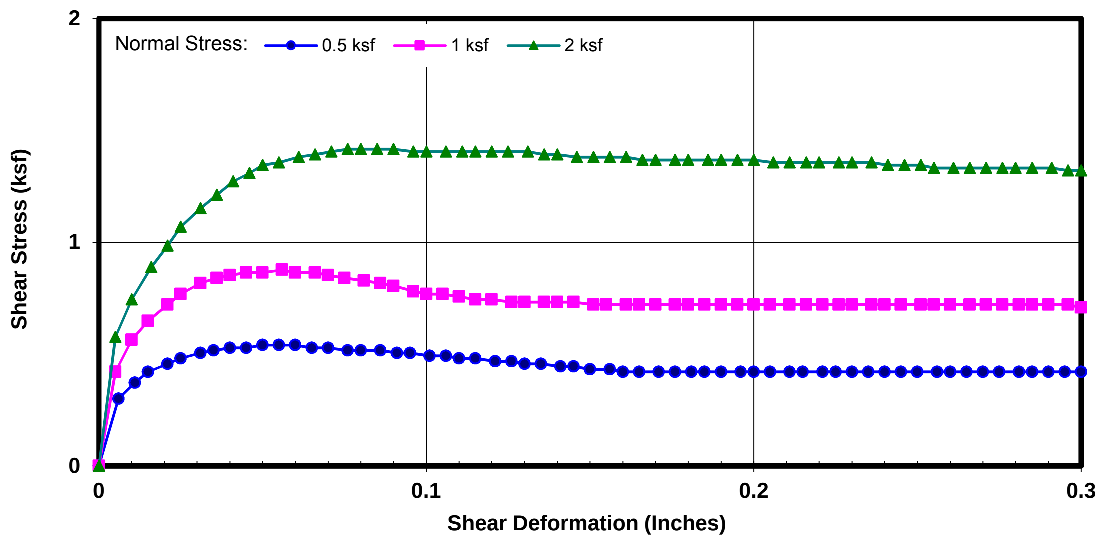
Figure B-5

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	Municipal Water District - West Valley Feeder	**Tested By:** ST	**Date:** 02/06/18
Project No.:	20180213.002A	**Computed By:** JP	**Date:** 02/07/18
Boring No.:	B-4	**Checked by:** AP	**Date:** 02/09/18
Sample No.:	2 **Depth (ft):** 5		
Sample Type:	Mod. Cal.		
Soil Description:	Silty Sand, fine-grained		
Test Condition:	Inundated **Shear Type:** Regular		

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
121.5	115.2	5.5	17.0	32	99	0.5	0.540	0.420
						1	0.876	0.708
						2	1.416	1.320



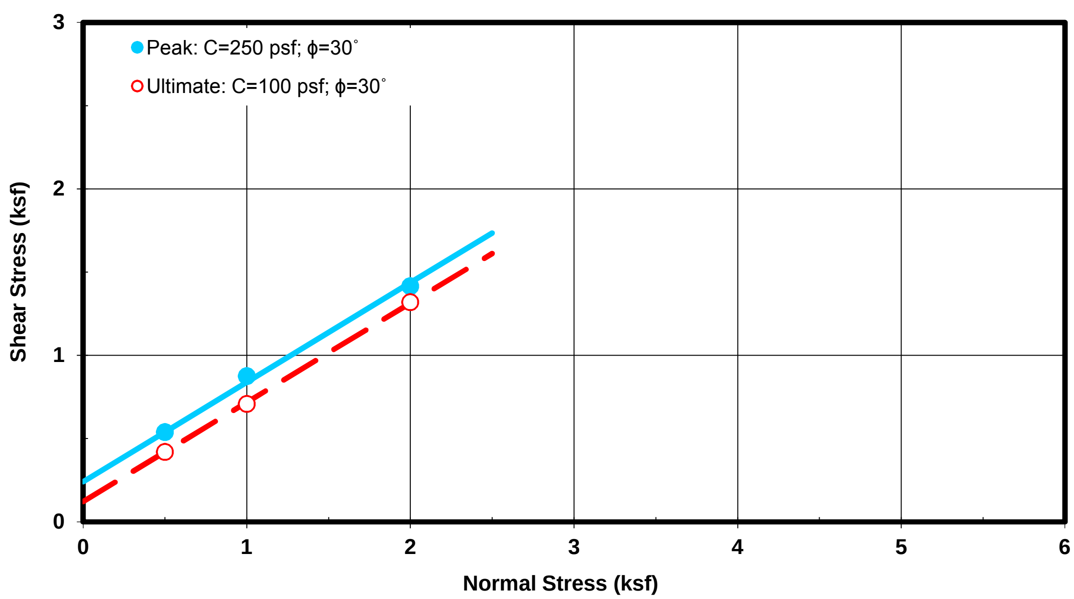


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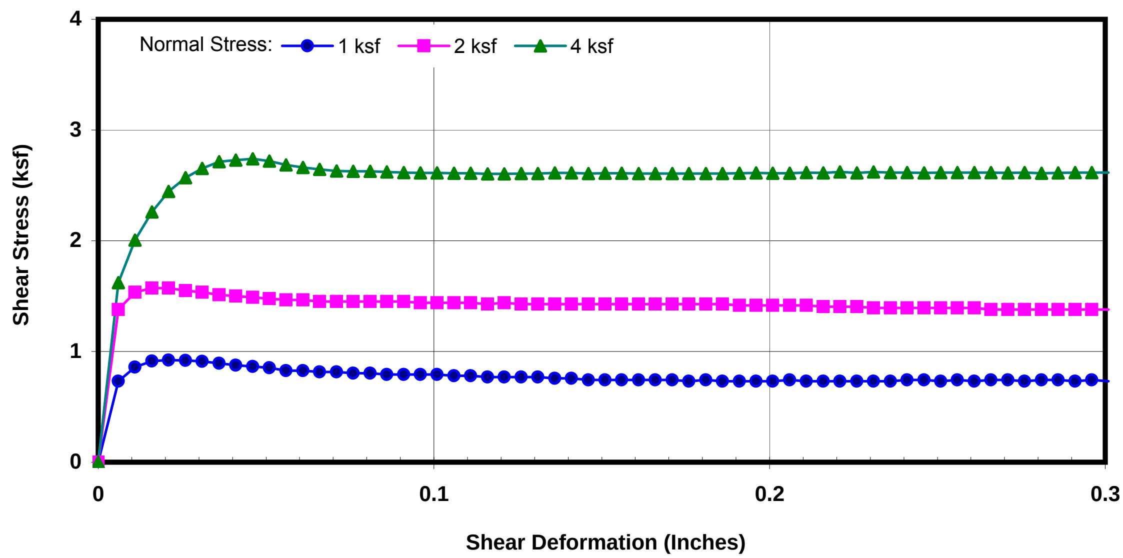
Figure B-6

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	Municipal Water District - West Valley Feeder	**Tested By:**	LS	**Date:**	02/26/18
Project No.:	20180213.002A	**Computed By:**	JP	**Date:**	02/27/18
Boring No.:	B-5	**Checked by:**	AP	**Date:**	02/27/18
Sample No.:	1 **Depth (ft):** 0-5				
Sample Type:	Remolded to 90% RC at opt. MC				
Soil Description:	Silty Sand				
Test Condition:	Inundated **Shear Type:** Regular				

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
126.6	117.3	7.9	14.6	49	90	1	0.923	0.732
						2	1.572	1.380
						4	2.739	2.616



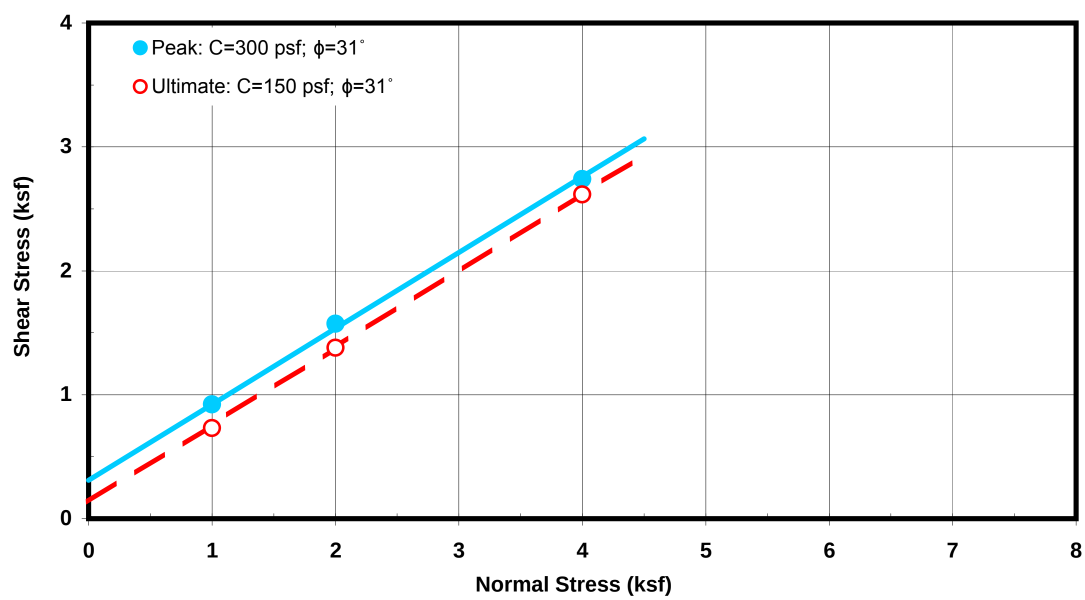


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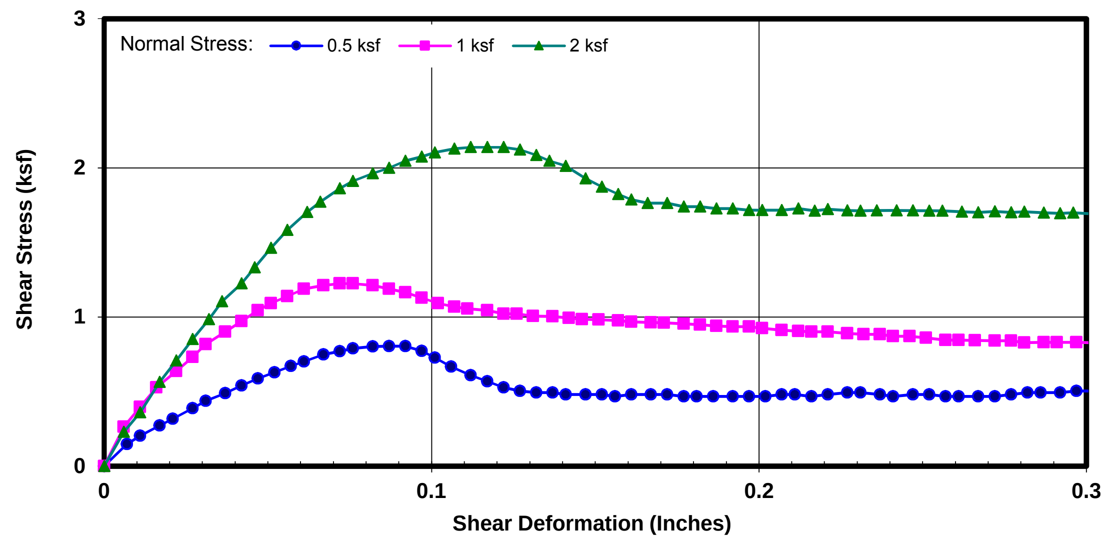
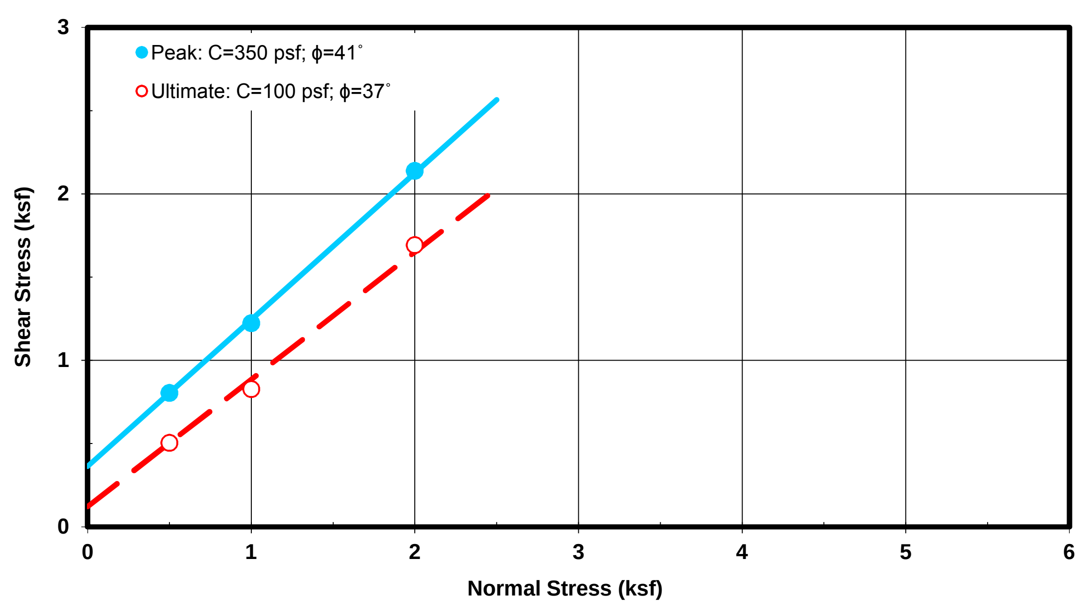
Figure B-7

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Municipal Water District - West Valley Feeder
Tested By: ST **Date:** 02/06/18
Project No.: 20180213.002A
Computed By: JP **Date:** 02/07/18
Boring No.: B-5
Checked by: AP **Date:** 02/09/18
Sample No.: 3 **Depth (ft):** 5
Sample Type: Mod. Cal.
Soil Description: Silty Sand
Test Condition: Inundated **Shear Type:** Regular

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
125.0	118.2	5.7	15.5	36	98	0.5	0.804	0.504
						1	1.224	0.828
						2	2.138	1.692





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APPENDIX C
Seismic Refraction Survey Report

ADVANCED GEOSCIENCE, INC.
Geology and Geophysics
Subsurface Exploration Non-Destructive Evaluation

24701 Crenshaw Blvd.
Torrance, California
90505 USA
Telephone (310) 378-7480
Fax (310) 872-5323
www.AdvancedGeoscience.com

February 6, 2018
via. Email (4 pages + Attachments)

Kleinfelder, Inc.
2280 Market Street
Suite 300
Riverside, California 92501

Attention: Mr. Richard Escandon, PG, CEG

Re: **Summary Report**
Seismic Refraction Survey for Bedrock Investigation
At MWD West Valley Feeders
Chatsworth, California

1.0 INTRODUCTION

This report summarizes the seismic refraction survey completed by Advanced Geoscience, Inc. at referenced site. This survey recorded the arrival times of seismic waves generated at the ground surface to prepare subsurface seismic velocity profiles for investigation of bedrock structure and rippability. The survey was performed along seismic survey lines positioned across the area shown on the site map in Figure 1 where grading is proposed for a future road.

The seismic refraction tomography data were recorded by Advanced Geoscience during a one-day field program completed on January 11, 2018. The data were recorded along two survey lines designated as Lines 1 and 2 (Figure 1). The data underwent computer processing to prepare 2D subsurface profiles showing seismic compressional-wave velocity layering in the upper 40 feet.

The following sections of this report provide a summary of our field survey procedures and methods of data processing and evaluation. A concluding section discusses the results of this seismic velocity profiling and compares these estimated subsurface velocities to the range of rippabilty for various Caterpillar ripping equipment.

2.0 FIELD SURVEY

Advanced Geoscience set up two survey lines designated as Lines 1 and 2. Line 1 was positioned across the proposed grading area along a south-to-north traverse extending across a hillside (Figure 1). Line 2 was positioned along a northwest-to-southeast traverse along a trail leading to Line 1. Both survey lines were positioned along straight-line traverses set up to avoid the heavier brush.

The seismic data were recorded using a multi-channel Seistronix EX-6 data acquisition system. This recording system was connected to geophones (seismic motion detectors) positioned in the ground at 10-foot intervals along the survey lines. Lines 1 and 2 were both set up with 21 geophones to provide a total line length of 200 feet. The geophones were 4-Hertz (lower-cutoff frequency), vertically-aligned velocity transducers.

The refraction data were recorded from eleven seismic energy "source points" positioned along each survey line. The source points started 5 feet off the first geophone position and continued at 20 to 30-foot intervals between the geophone positions. The last source point was positioned 5 feet off the last geophone position.

The seismic energy was generated using a 20-pound sledge hammer. The sledge hammer was used to make three impacts on a metal plate placed on the ground surface. At each source point, the recordings from the impacts were summed together to increase the amplitude of the seismic wave arrivals.

The positions of Lines 1 and 2 were marked by stakes placed at the end points of the lines and various breaks in the topography along the lines. The Metropolitan Water District (MWD) later arranged for a survey crew to measure the coordinates and elevations of these stakes.

3.0 DATA PROCESSING AND EVALUATION

The seismic data quality was good and adequate for the purposes of this investigation. The field records showed seismic wave arrivals from subsurface refraction events at all of the geophone positions.

The field records were input into the RAYFRACT seismic refraction tomography software developed by Intelligent Resources, Inc. (www.rayfract.com). RAYFRACT was used to generate seismic compressional-wave velocity profiles. This refraction tomography modeling procedure is generally more capable of imaging sharper lateral velocity variations due to bedrock structure than other refraction data modeling methods.

RAYFRACT was first used to graphically pick first arrival times ("first breaks") for refracted waves traveling through the surface layer and into deeper higher-velocity layers. These time-distance data were used together with the geophone coordinates and elevations to conduct refraction tomography imaging of the subsurface seismic velocity layering. RAYFRACT first used the Delta TV (turning ray-based) method to generate an initial 2D velocity-depth model. This initial model was then refined to produce a closer fit to the arrival time data using the Wavepath Eikonal Traveltime (WET) tomographic inversion method with 25 iterations with a maximum velocity 3,500 m/sec. The best-fit velocity-depth models were then gridded and color contoured with SURFER (written by Golden Software, Inc.) to show estimated vertical and lateral velocity variations.

Figures 2 and 3 show the resulting seismic compressional-wave velocity profiles for Lines 1 and 2.

4.0 DISCUSION OF RESULTS

The seismic compressional-wave velocity profiles for Lines 1 and 2 show 2,000 ft/sec or lower velocity layering in the upper 5 to 10 feet below ground surface (BGS). The materials in this depth interval are mostly colluvial soils and unconsolidated, decomposed bedrock. Below this depth the 3,000+ ft/sec velocity layering probably represents the upper weathered surface of the intact bedrock, which is mapped in this area as the late Cretaceous, Chatsworth Formation sandstone (reference: Preliminary Geologic Map of Los Angeles Quadrangle, USGS Open-File Report 2005-1019). Below this depth the bedrock velocities increase. Line 1 shows bedrock velocities as high as 8,000 ft/sec at the 40-foot depth level. Line 2 shows lower velocities in the range 5,000 to 5,500 ft/sec at the 40-foot depth level.

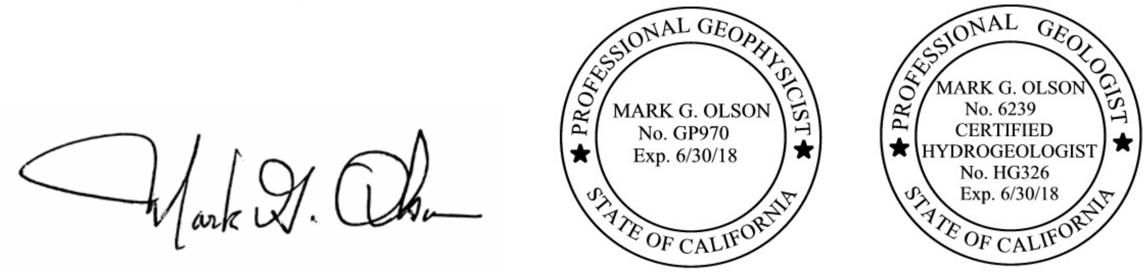
We understand that the depth of grading for the proposed road in this area is less than 20 feet BGS. The seismic velocities estimated along Lines 1 and 2 for this 20-foot depth interval are less than 6,000 ft/sec which indicates this upper bedrock material is mostly rippable for the Caterpillar D8R through D11R grading equipment. Figures 2 and 3 display the seismic velocity ranges for the rippability of sandstone bedrock estimated based on the graphs in the Caterpillar Handbook of Ripping, 12th Edition (Caterpillar, Inc., 2000). These velocity ranges are shown superimposed on the color velocity scales for the compressional-wave velocity profiles for Lines 1 and 2.

____

Advanced Geoscience appreciates this opportunity to be of service to Kleinfelder and the Metropolitan Water District. If you have any questions or additional requests concerning this seismic refraction survey please contact the undersigned.

Sincerely,

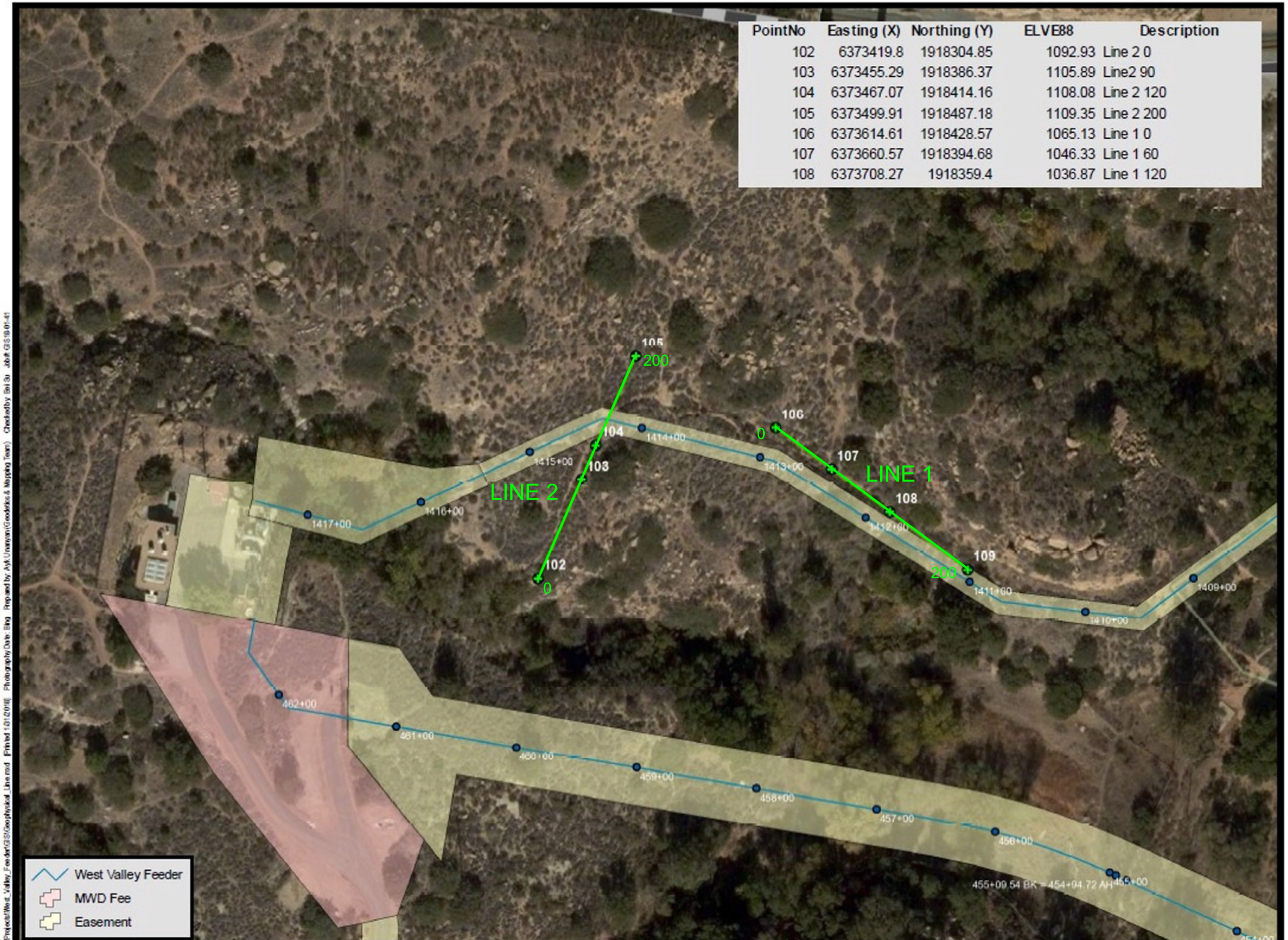
Advanced Geoscience, Inc.



Mark G. Olson, PGp, PG, CHG
Advanced Geoscience, Inc.
Principal Geophysicist

Attachments:

Figure 1	Site Plan Showing Seismic Survey Lines 1 and 2
Figure 2	Line 1- Seismic Refraction Compressional-Wave Velocity Profile
Figure 3	Line 2- Seismic Refraction Compressional-Wave Velocity Profile



PointNo	Easting (X)	Northing (Y)	ELVE88	Description
102	6373419.8	1918304.85	1092.93	Line 2 0
103	6373455.29	1918386.37	1105.89	Line2 90
104	6373467.07	1918414.16	1108.08	Line 2 120
105	6373499.91	1918487.18	1109.35	Line 2 200
106	6373614.61	1918428.57	1065.13	Line 1 0
107	6373660.57	1918394.68	1046.33	Line 1 60
108	6373708.27	1918359.4	1036.87	Line 1 120

The Metropolitan Water District of Southern California
Engineering Services Group

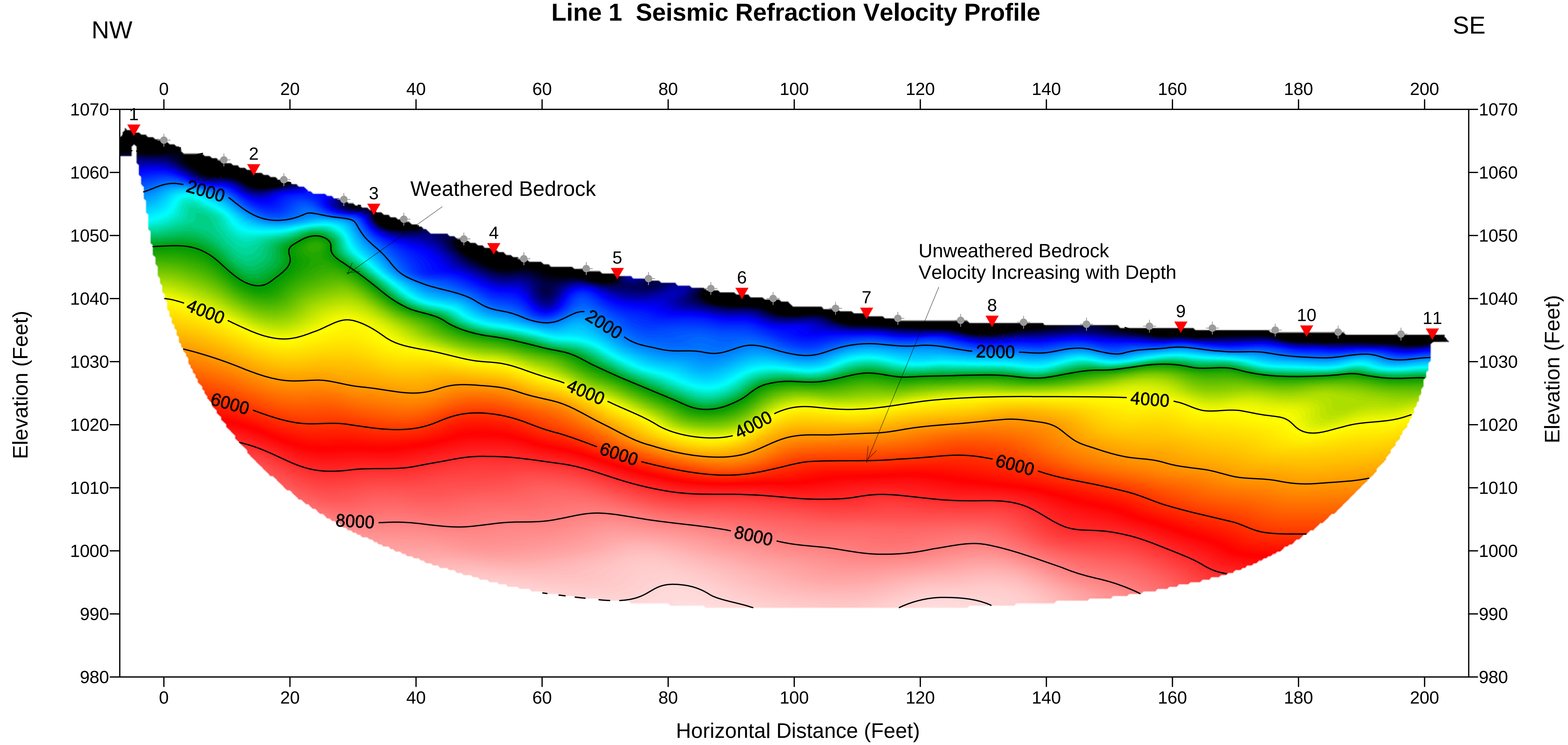
West Valley Feeders
Survey Data - Geophysical Line

Feet 0 25 50 100

Site Map Showing Seismic Survey Lines 1 and 2
MWD West Valley Feeders Chatsworth, CA

Figure 1
Advanced Geoscience, Inc.

Line 1 Seismic Refraction Velocity Profile



Horizontal & Vertical Scale 1 inch= 20 Feet
Seismic Velocity Contour Interval 1,000 ft/sec

Based on RAYFRACT Refraction Tomography
Initial Delta TV Velocity Model + 25 WET Iterations w/Vmax= 3,500 m/sec

Estimated Rippability for Caterpillar Equipment

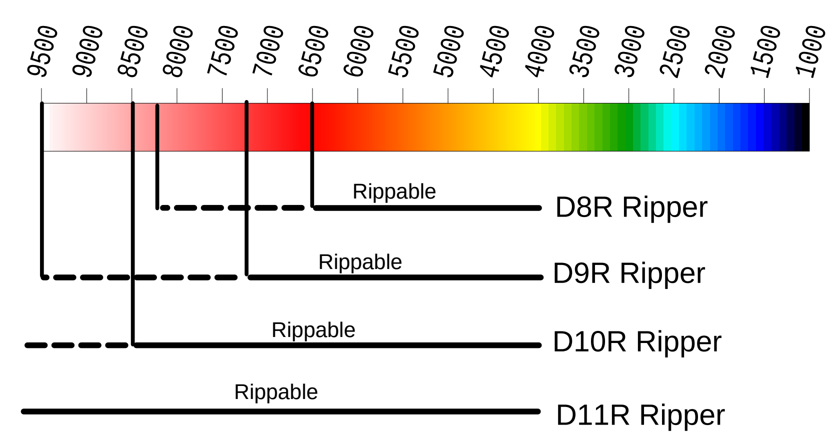
Based on Caterpillar Handbook of Ripping, 12th Edition

Rippable

Marginal Rippability

Seismic Velocity Range for Rippability of
Sandstone Rock Type for
Chatsworth Formation Sandstone (Kc)

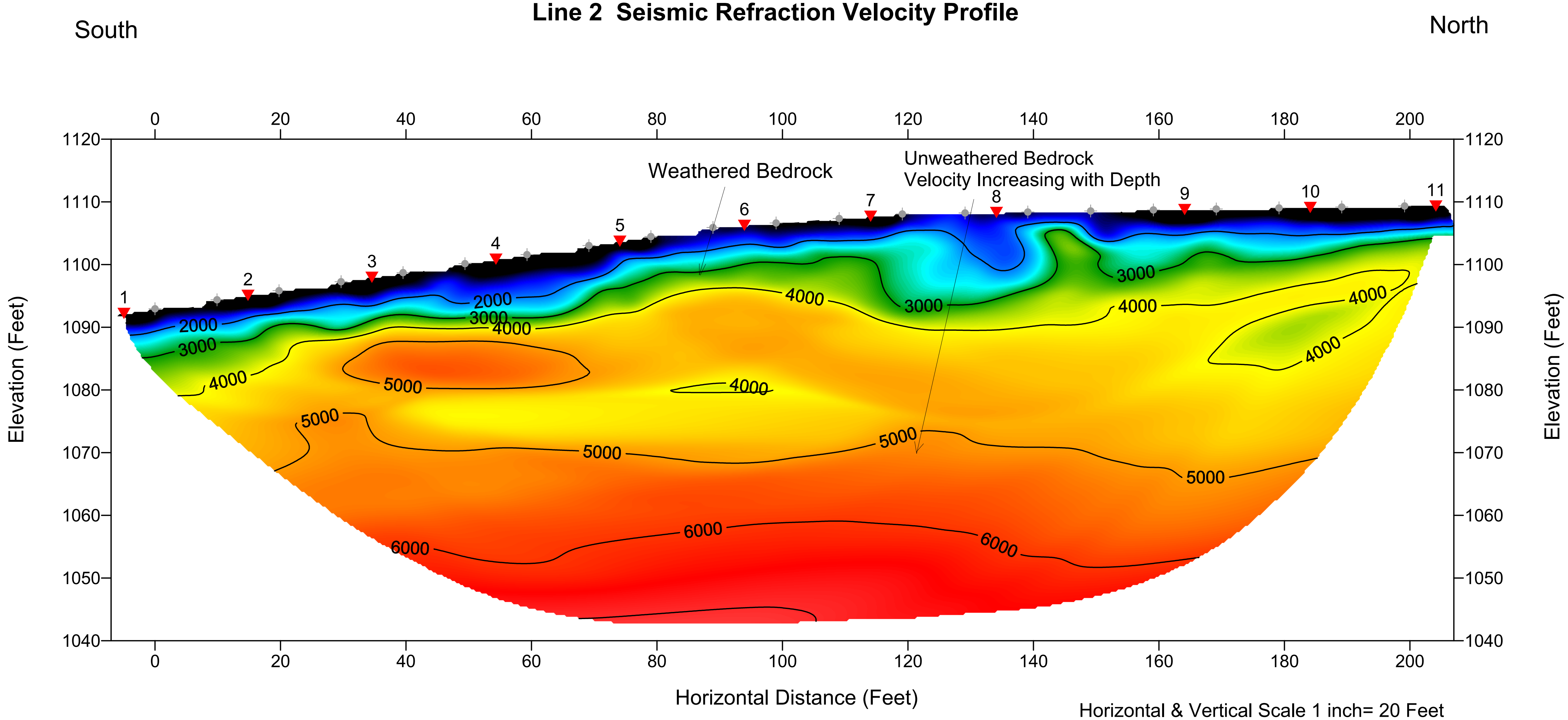
Estimated Seismic Compressional Wave Velocity (ft/sec)



Line 1 Seismic Refraction Velocity Profile
For Bedrock Investigation at MWD West Valley Feeders
Chatsworth, California

Figure 2
Advanced Geoscience, Inc.

Line 2 Seismic Refraction Velocity Profile



Horizontal & Vertical Scale 1 inch= 20 Feet
Seismic Velocity Contour Interval 1,000 ft/sec

Based on RAYFRACT Refraction Tomography
Initial Delta TV Velocity Model + 25 WET Iterations w/Vmax= 3,500 m/sec

Estimated Rippability for Caterpillar Equipment

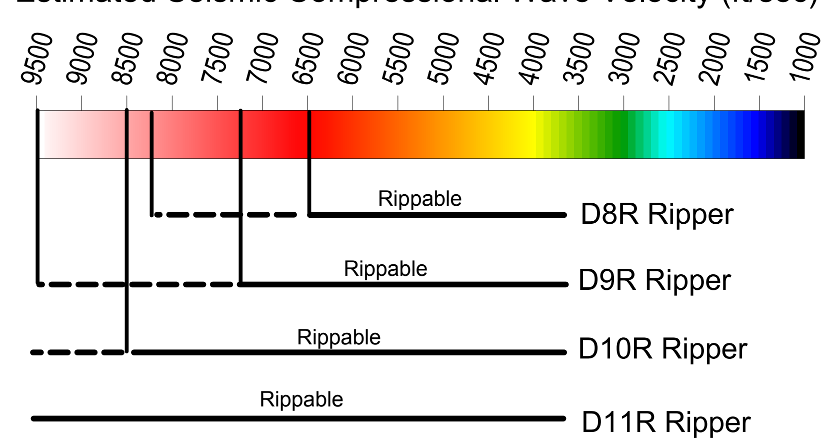
Based on Caterpillar Handbook of Ripping, 12th Edition

Rippable

Marginal Rippability

Seismic Velocity Range for Rippability of
Sandstone Rock Type for
Chatsworth Formation Sandstone (Kc)

Estimated Seismic Compressional Wave Velocity (ft/sec)



Line 2 Seismic Refraction Velocity Profile
For Bedrock Investigation at MWD West Valley Feeders
Chatsworth, California

Figure 3
Advanced Geoscience, Inc.

APPENDIX G

PALEONTOLOGICAL RECORDS SEARCH

Natural History Museum
of Los Angeles County
900 Exposition Boulevard
Los Angeles, CA 90007

tel 213.763.DINO
www.nhm.org

Vertebrate Paleontology Section
Telephone: (213) 763-3325

e-mail: smcleod@nhm.org

12 July 2018

Psomas
3 Hutton Centre Drive, Suite 200
Santa Ana, CA 92707-8794

Attn: Melissa Macias, Paleontologist

re: Paleontological Resources for the proposed West Valley Feeder Project, Psomas Project 3MWD010204, near the Chatsworth Reservoir, Los Angeles County, project area

Dear Melissa:

I have conducted a thorough search of our Vertebrate Paleontology records for the proposed West Valley Feeder Project, Psomas Project 3MWD010204, near the Chatsworth Reservoir, Los Angeles County, project area as outlined on the portion of the Oat Mountain USGS topographic quadrangle map that you sent to me via e-mail on 9 July 2018. We have no vertebrate fossil localities that lie directly within the boundaries of the proposed project area, but we do have localities nearby from the same sedimentary deposits that occur in the proposed project area.

In the entire proposed project area there are exposures of the marine late Cretaceous Chatsworth Formation. Our closest vertebrate fossil localities from the Chatsworth Formation are LACM 4913-1914, southwest of the proposed project area on the south side of Dayton Canyon, that produced fossil shark specimens including sand sharks, Carcharhiniformes, mackerel shark, *Cretolamna appendiculata*, crow shark, *Squalicorax kaupi*, dogfish shark, *Squalus*, and angel shark, *Squatina hassei*. Specimens of all of these sharks from localities LACM 4913-4914 were figured in the scientific literature by Welton and Alderson (1981. A Preliminary Note on the Late Cretaceous Sharks of the Chatsworth Formation at Dayton Canyon, Simi Hills, Los Angeles County, California. Society of Economic Paleontologists & Mineralogists Guidebook, 1981).

Any excavations in the Chatsworth Formation exposed throughout the proposed project area may well encounter significant remains of fossil vertebrates. Any substantial excavations in the proposed project area, therefore, should be monitored closely to quickly and professionally recover any fossil remains while not impeding development. Also, sediment samples should be collected and processed to determine the small fossil potential in the proposed project area. Any fossils collected should be placed in an accredited scientific institution for the benefit of current and future generations.

This records search covers only the vertebrate paleontology records of the Natural History Museum of Los Angeles County. It is not intended to be a thorough paleontological survey of the proposed project area covering other institutional records, a literature survey, or any potential on-site survey.

Sincerely,

Samuel A. McLeod

Samuel A. McLeod, Ph.D.
Vertebrate Paleontology

enclosure: invoice