Appendix G
Geological Reconnaissance

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GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

GEOLOGIC RECONNAISSANCE

FRANKLIN RIDGE ROAD EXTENSION
SERRA MESA COMMUNITY
SAN DIEGO, CALIFORNIA

PREPARED FOR

DUDEK
ENCINITAS, CALIFORNIA

JUNE 5, 2013
REVISED JULY 22, 2013
PROJECT NO. G1535-52-01

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G1535-52-01
June 5, 2013
Revised July 22, 2013

Dudek
605 Third Street
Encinitas, California 92024

Attention: Mr. Shawn Shamlou

Subject: GEOLOGIC RECONNAISSANCE
FRANKLIN RIDGE ROAD EXTENSION
SERRA MESA COMMUNITY
SAN DIEGO, CALIFORNIA

Dear Mr. Shamlou:

In accordance with your request and authorization of our proposal No. LG-12123 dated April 26, 2012, we have performed a geologic reconnaissance within the area of proposed roadway construction associated with the Franklin Ridge Road Extension project. The accompanying report presents the findings of our study and our preliminary recommendations relative to the geotechnical aspects of designing improvements as presently proposed.

Should you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

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TABLE OF CONTENTS

GEOLOGIC RECONNAISSANCE

1. PURPOSE AND SCOPE

This report presents the results of a geologic reconnaissance for proposed improvements and new construction associated with the Franklin Ridge Road Extension project. The site is located west of I-805 and south of the existing Phyllis Place in the Serra Mesa Community area of San Diego, California (see Vicinity Map, Figure 1). The purpose of this study is to evaluate the existing geologic conditions and the geologic/geotechnical hazards that will likely affect the property. We expect this report can be used for the environmental impact report (EIR) and tentative map submittals. A geotechnical investigation should be performed to provide final engineering recommendations that would include performing fieldwork, laboratory testing, and engineering analyses prior to submittal of construction documents.

The scope of this geologic reconnaissance included a review of readily available published and unpublished geologic literature, performing a limited field investigation, laboratory testing and preliminary engineering analyses, and preparing this report. Additionally, we reviewed previous geotechnical investigations prepared by Geocon Incorporated for the adjacent Quarry Falls (Civita) project, (see List of References). This report summarizes our findings and conclusions regarding the geologic conditions at the site and our recommendations for future geotechnical studies.

We performed a limited field investigation on May 10, 2013, that included excavating 4 hand test pits to a maximum depth of approximately 2½ feet. The approximate locations of the hand dug test pits are presented on the *Geologic Map*, Figure 2. We encountered refusal at shallow depths on gravel and cobbles present within the surficial materials. We tested selected soil samples obtained during the limited field investigation to evaluate pertinent physical properties for engineering analyses and to assist in providing preliminary recommendations for proposed grading and roadway construction. Details of the laboratory tests and a summary of the test results are presented in Appendix A.

We prepared the Geologic Map (Figure 2), based on the project plan titled *Franklin Ridge Road Extension Project Components*, prepared by Dudek, and the recent grading data and Geologic Map for Unit F prepared by Geocon Incorporated from the Quarry Falls Project (Civita). The map depicts the proposed new roadway alignment, existing topography, preliminary finish grade elevations, and mapped geologic contacts based on our reconnaissance. The conclusions and preliminary recommendations presented herein are based on an analysis of the data reviewed as part of this study and our experience with similar soil and geologic conditions.

2. SITE AND PROJECT DESCRIPTION

The property is located west of I-805 and south of the existing Phyllis Place in the Serra Mesa Community area of San Diego, California. The site currently consists of mostly undeveloped open space. A City of San Diego storm drain flows to the southwest to an existing storm drain structure constructed during the adjacent grading operation. Additionally, SDG&E overhead electrical easement and a high-pressure gas line traverse the northern portion of the proposed roadway extension. We also observed a fiber optic utility easement present extending parallel to Phyllis Place approximately 10 feet south of the back of curb. The key topography features of the site consist of a drainage channel and sloping terrain. The surface of the site ranges from an elevation of about 225 feet above Mean Sea Level (MSL) within the lower drainage to about 290 feet MSL along Phyllis Place.

We understand the project alignment extends approximately 500 feet from Phyllis Place in Serra Mesa south to the northern boundary of the Mission Valley Community. Specifically, the proposed roadway is a four-lane major arterial with an approximately 120-foot right-of-way and bicycle and pedestrian facilities, extending from a signalized intersection at Phyllis Place to the Quarry Falls Development. Significant amounts of fill on the order of 60 feet in height with side slopes of 2H:1V and 3H:1V are expected to establish the proposed roadway embankment.

The locations and descriptions of the site and proposed improvements are based on a site reconnaissance and a review of the referenced tentative map. If development plans differ significantly from those described herein, Geocon Incorporated should be consulted for review and possible revisions to this report.

3. SOIL AND GEOLOGIC CONDITIONS

Based on four shallow hand excavated test pits, review of readily available published and unpublished geologic literature, we expect five surficial soil types and one geologic formation underlie the project site. The surficial deposits consist of compacted fill, undocumented fill, topsoil, alluvium, and Terrace Deposits underlain by the Stadium Conglomerate. Each of the surficial soil types and the geologic unit expected is described below in order of increasing age. With the exception of topsoil (unmapped), the approximate extent of the surficial deposits is shown on the Geologic Map, Figure 2. The subsurface materials encountered in our test pits are summarized in Table 3.

TABLE 3
SUMMARY OF SUBSURFACE MATERIALS ENCOUNTERED AT TEST PIT LOCATIONS

Test Pit No.	Depth (feet)	USCS Soil Classification	Description
TP-1	0-2½	SM	Light yellowish brown, Silty, fine to medium SAND
TP-2	0-2½	SM	Light yellowish brown, Silty, fine to medium SAND
TP-3	0-2	CL	Dark reddish brown, Sandy CLAY
TP-4	0-2½	CL	Dark reddish brown, Sandy CLAY

3.1 Compacted Fill (Qcf)

Compacted fill associated with the adjacent grading operations are present along the western margins of the proposed roadway. The fill placed was tested and observed by Geocon Incorporated. In general, the compacted fill consists of sand, silt, and clay derived from on-site excavations and is suitable for the support of the proposed compacted fill and additional structural loads.

3.2 Undocumented Fill (Qudf)

We expect the northern portion of the proposed roadway is underlain by undocumented fill that is likely associated with the original construction of Phyllis Place. Based on a review of the referenced aerial photos, the existing drainage channel was extended northeasterly prior to achieving existing grades. We estimate the maximum thickness of the undocumented fill to be approximately 70 feet. The undocumented fill is likely to consist of silty sand to sandy silt with gravel and cobble. Portions of the undocumented fill will likely require remedial grading in areas that will receive additional fill and/or settlement-sensitive improvements, where possible. Further geotechnical evaluation of the undocumented fill will be necessary to assess the in-situ conditions of the undocumented fill.

3.3 Topsoil (unmapped)

We expect the Stadium Conglomerate is overlain by thin veneer of topsoil. Based on our test pits TP-3 and TP-4, the topsoil was generally 2 to 3 feet thick and composed of clayey sand and silty to sandy clay with abundant gravel and cobble. The topsoil could be highly expansive and will require removal in areas that will receive additional fill and/or settlement-sensitive improvements.

3.4 Alluvium (Qal)

Based on review of the referenced report and our field observations, alluvial soil exists within the drainage channel. These deposits typically consist of medium dense, silty, fine to coarse sand with abundant gravel and cobble. We expect the alluvium possesses a thickness of 6 to 8 feet. The alluvium is subject to consolidation settlement and is not suitable for the support of structural fill and settlement-sensitive structures.

3.5 Terrace Deposits (Qt)

Based on review of the referenced report and our field observations, Terrace Deposits underlie the topsoil and are exposed on the existing cut slope to the west of the proposed roadway. It is likely that Terrace Deposits will not be encountered during grading operations. These deposits have been mapped as old alluvium by Kennedy and Tan 2005 (*Geologic Map of the San Diego 30' x 60' Quadrangle, California*, 2005). The Terrace Deposits are composed of dense silty sand and hard siltstone. The Terrace Deposits, if encountered, are relatively dense and considered suitable for support of structural improvements. Surficial slope protection may be needed to reduce erosion if cohesionless sand zones are exposed on cut slopes.

3.6 Stadium Conglomerate (Tst)

The Eocene-age Stadium Conglomerate is the predominant formational unit on the site. This unit was the primary material mined to generate aggregate. In general, the Stadium Conglomerate consists of a dense to very dense, yellow to light brown, cobble conglomerate. The deposit contains a relatively high percentage of rounded cobble (up to approximately 60 percent by weight) embedded in a silty to clayey, fine to medium sand soil matrix. The cobble typically ranges in size from approximately 3 inches to 12 inches. When excavated, the Stadium Conglomerate typically consists of *low* to *very low* expansive silty/clayey sands that possess good shear strength characteristics in either a natural or properly compacted condition. The Stadium Conglomerate is suitable for support of additional fill and structural loading.

4. GROUNDWATER

Groundwater is expected to be deep but perched groundwater may be near the water level within the existing drainage channel. Groundwater elevations are dependent on seasonal precipitation, irrigation, and land use among other factors and, vary as a result. Proper surface drainage will be important to future performance of the project.

5. GEOLOGIC HAZARDS

5.1 Geologic Hazard Category

The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Map Sheet 21 defines the site with a Hazard Category 53: *Level or sloping terrain, unfavorable geologic structure, low to moderate risk.*

5.2 Faulting and Seismicity

A review of geologic literature and experience with the soil and geologic conditions in the general area indicate that known active, potentially active, or inactive faults are not located at the site. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,000 years. The site is not located within a State of California Earthquake Fault Zone.

According to the computer program *EZ-FRISK* (Version 7.62), six known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database that provides several models and combinations of fault data to evaluate the fault information. The nearest known active faults are the Newport-Inglewood/Rose Canyon Fault system, located approximately 3 miles west of the site and is the dominant source of potential ground motion. Earthquakes that might occur on the Rose Canyon Fault Zone or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood Fault are 7.5 and 0.46g, respectively. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Rose Canyon Fault are 6.9 and 0.40g, respectively. Table 5.2.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for these and other faults in relationship to the site location. We used acceleration attenuation relationships developed by Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2008) NGA acceleration-attenuation relationships in our analysis.

TABLE 5.2.1
DETERMINISTIC SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2008 (g)
Newport-Inglewood	3	7.5	0.37	0.38	0.46
Rose Canyon	3	6.9	0.32	0.37	0.40
Coronado Bank	16	7.4	0.18	0.14	0.17
Palos Verdes Connected	16	7.7	0.20	0.15	0.20
Elsinore	38	7.9	0.12	0.08	0.10
Earthquake Valley	43	6.8	0.06	0.05	0.04

It is our opinion the site could be subjected to moderate to severe ground shaking in the event of an earthquake along any of the faults listed in Table 5.2.1 or other faults in the southern California/ northern Baja California region. We do not consider the site to possess a greater risk than that of the surrounding developments.

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each mapped Quaternary fault is proportional to the faults slip rate. The program accounts for earthquake magnitude as a function of fault rupture length, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) in the analysis. Table 5.2.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 5.2.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2008 (g)
2% in a 50 Year Period	0.45	0.48	0.52
5% in a 50 Year Period	0.30	0.31	0.34
10% in a 50 Year Period	0.20	0.21	0.022

The California Geologic Survey (CGS) has a program that calculates the ground motion for a 10 percent of probability of exceedence in 50 years based on an average of several attenuation relationships. Table 5.2.3 presents the calculated results from the *Probabilistic Seismic Hazards Mapping Ground Motion* Page from the CGS website.

TABLE 5.2.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Soft Rock	Calculated Acceleration (g) Alluvium
0.27	0.29	0.33

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be evaluated in accordance with the 2010 California Building Code (CBC) guidelines or guidelines currently adopted by the City of San Diego.

5.3 Ground Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects that earth surface. The potential for ground rupture is considered to be very low due to the absence of active faults at the subject site.

5.4 Seiches and Tsunamis

A tsunami is a series of long-period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The first-order driving force for locally generated tsunamis offshore southern California is expected to be tectonic deformation from large earthquakes (Legg, *et al.*, 2002). Historically, tsunami wave heights have ranged up to 3.7 feet in the San Diego area (URS, 2004). The County of San Diego Hazard Mitigation Plan maps zones of high risk for tsunami run-up for coastal areas throughout the county. The site is not included within one of these hazard areas. The site is located about 5 miles from the Pacific Ocean at a minimum elevation of approximately 240 feet above MSL. Therefore, the risk of tsunamis affecting the site is negligible.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located in the vicinity of or downstream from such bodies of water. Therefore, the risk of seiches affecting the site is negligible.

5.5 Liquefaction Potential

Liquefaction typically occurs during seismic shaking in relatively loose, cohesionless soil that exists below the groundwater surface. Under these conditions, a seismic event could result in a rapid pore water pressure increase from the earthquake-generated ground accelerations. The potential for liquefaction at the site is considered low due to the presence of shallow dense formational materials and the lack of permanent, near-surface groundwater.

5.6 Landslides

Examination of aerial photographs in our files, review of published geologic maps for the site vicinity, and the relatively level topography, it is our opinion that landslides are not present at the property or at a location that could impact the subject site.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 From a geotechnical engineering standpoint, it is our opinion that the site is suitable for the proposed development, provided the recommendations of this report and future geotechnical investigations are followed.

6.1.2 The site is expected to be underlain by topsoil, compacted fill, undocumented fill, alluvium, and the Terrace Deposits overlying the Stadium Conglomerate. We expect the alluvium within the drainage channel to extend to a maximum depth of approximately 6 to 8 feet below the existing grade. We expect the maximum thickness of the undocumented fill to be approximately 70 feet.

6.1.3 We expect the planned roadway will be supported on compacted fill placed during the roadway grading operations.

6.1.4 Excavation of the surficial soil should generally be possible with moderate effort using conventional, heavy-duty equipment during grading and trenching operations.

6.1.5 The site is located approximately 3 miles from the nearest active fault, the Newport-Inglewood/Rose Canyon Fault system. Based on our background research, active, potentially active, or inactive faults do not extend across or trend toward the site. Risks associated with seismic activity at this site generally consist of the potential for strong seismic shaking. The site is not mapped in a High Liquefaction Hazard Zone as defined by the City of San Diego (2008).

6.1.6 Groundwater is expected to be deep but perched groundwater may be encountered near the water level in the drainage channel. Groundwater could have an influence on construction operations depending on the volume of perched groundwater, utility invert elevations, and excavation depths. Stabilization and/or dewatering may be necessary for excavations with seepage.

6.2 Soil and Excavation Characteristics

6.2.1 We expect the existing soil can be considered to be “non-expansive” and “expansive” (expansion index [EI] of 20 or less and greater than 20, respectively) as defined by 2010 California Building Code (CBC) Section 1803.5.3. Table 6.2 presents soil classifications based on the expansion index. Based on our laboratory testing the topsoil present onsite soil onsite possess a “very high” expansion potential (expansion index of 130 or greater).

TABLE 6.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

6.2.2 Surficial deposits can be excavated with moderate effort using conventional heavy-duty grading equipment. Gravel and cobbles are not uncommon within the Stadium Conglomerate and may require special excavation equipment if encountered. This issue may be the focus of future studies.

6.2.3 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, if improvements that could be susceptible to corrosion are planned, further evaluation by a corrosion engineer should be performed.

6.3 Preliminary Grading Recommendations

6.3.1 Grading should be performed in accordance with the attached *Recommended Grading Specifications* (Appendix B). Where the recommendations of this section conflict with Appendix B, the recommendations of this section take precedence. Earthwork should be observed and fill tested for proper compaction by Geocon Incorporated.

6.3.2 Prior to commencing grading, a preconstruction conference should be held at the site with the owner or developer, grading contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.

6.3.3 Site preparation should begin with the removal of deleterious material and vegetation. The depth of removal should be such that material exposed in cut areas or soils to be used as fill are relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.

6.3.4 In general, the loose and/or soft portions of surficial soil within areas of planned grading should be removed and properly compacted prior to placing additional fill and/or structural loads. The actual extent and depth of surficial soils requiring removal should be evaluated during the planned geotechnical investigation. Overly wet soils, as might be encountered in the vicinity of drainages, will require drying and/or mixing with drier soils to facilitate proper compaction.

6.3.5 Excavated, on-site soil if free of deleterious debris, expansive soil and large rock can be placed as fill and compacted in layers to the design finish grade elevations. Fill and backfill soil should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned as necessary, and compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. The upper 12 inches of soil beneath pavement areas should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content.

6.3.6 Import fill, should consist of granular materials with a “very low” to “low” expansion potential (EI less than 50) free of deleterious material or stones larger than 3 inches and should be compacted as recommended herein. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to evaluate its suitability as fill material.

6.4 Site Drainage and Moisture Protection

6.4.1 The existing drainage channel, storm drainpipe, and outlets should be mitigated as a part of the proposed site improvements via appropriate storm drain, subdrain, and/or canyon subdrain system.

6.4.2 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2010 CBC 1804.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Pavement drainage should be directed into conduits that carry runoff away from the proposed improvements.

6.4.3 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

6.4.4 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material.

6.4.5 If detention basins, bioswales, retention basins, or water infiltration devices are being considered, Geocon Incorporated should be retained to provide recommendations pertaining to the geotechnical aspects of possible impacts and design. Distress may be caused to planned improvements and properties located hydrologically downstream. The distress depends on the amount of water to be detained, its residence time, soil permeability, and other factors. We have not performed a hydrogeology study at the site. Downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other impacts as a result of water infiltration.

6.5 Future Geotechnical Investigation

6.5.1 A geotechnical investigation should be performed to drill 2 to 4 small diameter rotary wash borings to a depth of up to about 70 feet below the ground surface utilizing a truck-mounted drill rig, and perform 6 to 8 trenches utilizing a rubber tire backhoe. The field investigation would consist of sampling the soil conditions during excavation of the test borings, and trenches, to examine the soil conditions encountered, and evaluate the surficial deposits and depth of groundwater.

6.5.2 We should perform laboratory tests on selected soil samples to evaluate maximum dry density and optimum moisture content, shear strength, water-soluble sulfate content, consolidation, resistance value (R-Value), in-situ dry density and moisture content, and gradation of the soil encountered. Similar laboratory tests should also be performed on imported fill soil samples.

6.5.3 The geotechnical investigation report should present our findings, conclusions, and recommendations regarding the geotechnical aspects of grading and improvements as presently proposed. Excavation characteristics, geologic hazard analyses, and remedial grading measures at the site would be included in the report.

6.6 Preliminary Pavement Recommendations

6.6.1 We performed laboratory R-Value tests on one subgrade soil sample to provide preliminary recommendations for structural pavement sections for the subject roadway.

6.6.2 Public street paving should be designed in accordance with the City of San Diego Standard Drawing, Pavement Design Standard Schedule “J” when final Traffic Indices and R-Value test results of subgrade soil are completed. The results of our laboratory R-Value testing, as shown in Table A-II in Appendix A, indicate that the R-Value of the existing materials of approximately 21. For the purposes of preliminary design, we recommend to use an R-Value of between 20 to 29.9. The preliminary flexible pavement sections are presented in Table 6.6 below.

TABLE 6.6
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Street Classification	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Cement Treated Base (inches)
Major (4-Lane)	10.5	20 to 29.9	5.0	16.0

6.6.3 The upper 12 inches of the subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density based on ASTM D 1557 near to slightly above optimum moisture content beneath pavement sections.

6.7 Grading Plan Review

6.7.1 Geocon Incorporated should review the project grading plans prior to final design submittal to check if additional analysis and/or recommendations are required.

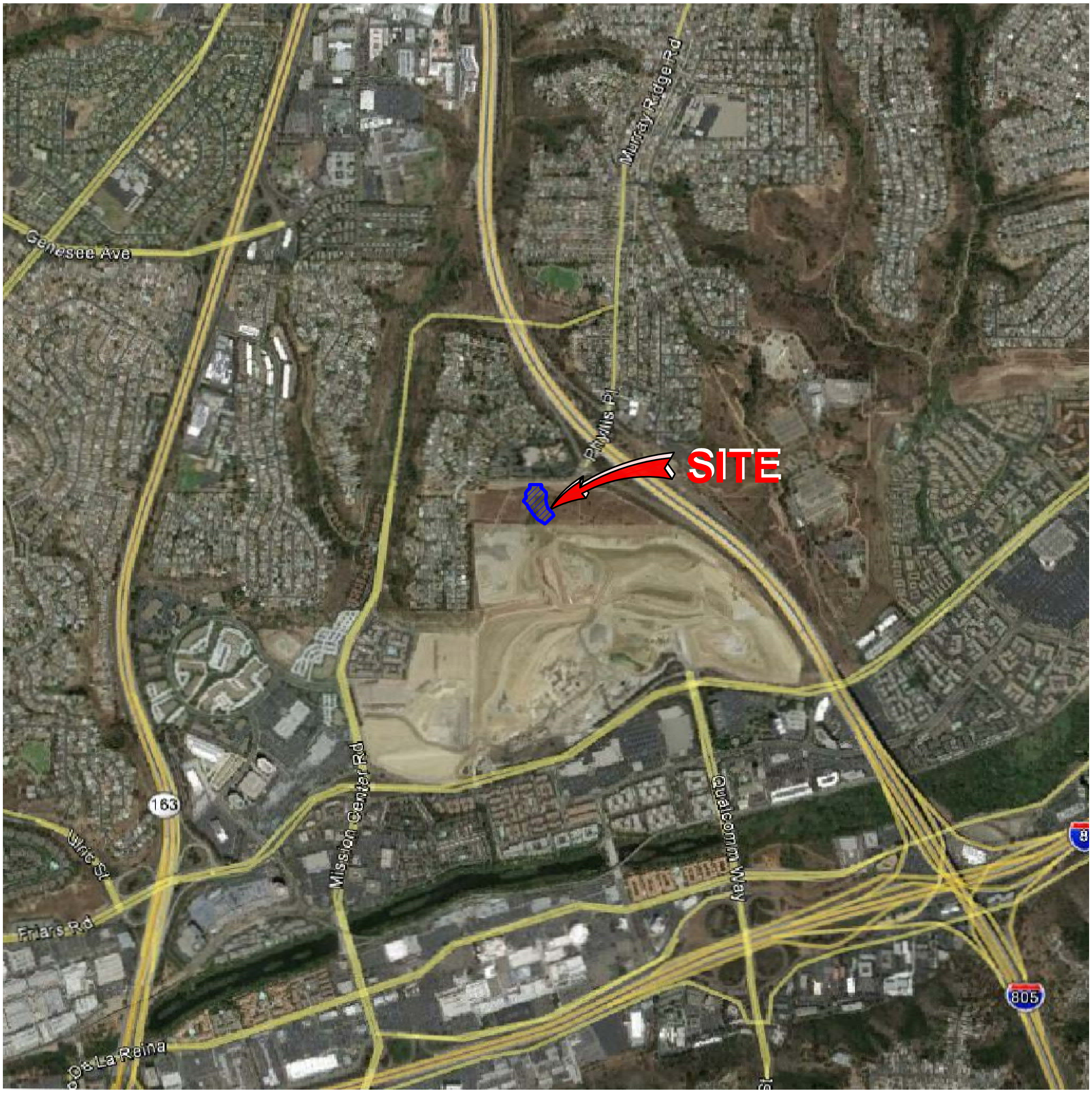
LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

2. This report is issued with the understanding that it is the responsibility of the owner or his representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.



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NO SCALE

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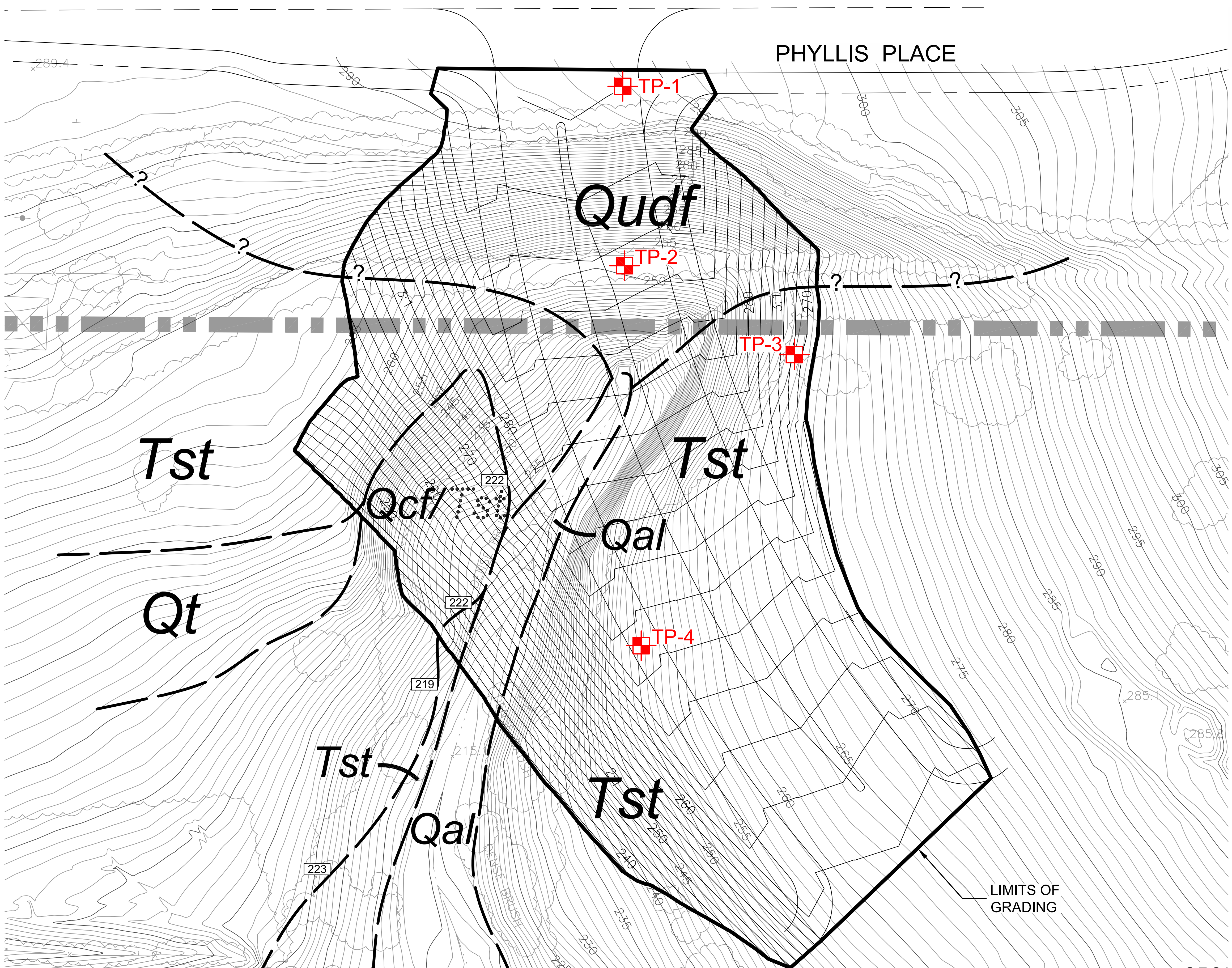
VICINITY MAP

FRANKLIN RIDGE ROAD EXTENSION
SAN DIEGO, CALIFORNIA

ME / AML		DSK/GTYPD

DATE 07 - 22 - 2013	PROJECT NO. G1535 - 52 - 01	FIG. 1

Vicinity Map



FRANKLIN RIDGE ROAD EXTENSION
SAN DIEGO, CALIFORNIA

0' 25' 50' 75' 100'
SCALE 1"= 50'

GEOCON LEGEND

Qcf ....COMPACTED FILL
Qudf ........UNDOCUMENTED FILL
Qal ....ALLUVIUM
Qt ....TERRACE DEPOSITS
Tst ....STADIUM CONGLOMERATE (Dotted Where Buried)
........APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
TP-4 ....APPROX. LOCATION OF HAND DUG TEST PIT
222 ....APPROX. ELEVATION AT BOTTOM OF FILL

GEOLOGIC MAP

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PROJECT NO. G1535 - 52 - 01
FIGURE 2
DATE 07 - 22 - 2013

APPENDIX A

APPENDIX A

LABORATORY TESTING

We performed laboratory testing to evaluate the physical and mechanical properties of the soil encountered at the site. We performed the laboratory tests in accordance with the current versions of the generally accepted *American Society for Testing Materials* (ASTM) procedures or other suggested procedures. We tested selected soil samples for their expansion index, and resistance value (R-Value). The results of our laboratory tests are presented in Tables A-I and A-II.

TABLE A-I
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	Expansion Classification	2010 CBC Classification
	Before Test	After Test				
TP-3	16.4	36.4	91.2	132	Very High	Expansive

TABLE A-II
SUMMARY OF LABORATORY RESISTANCE VALUE (R-VALUE) TEST RESULTS
ASTM D 2844

Sample No.	Laboratory R-Value Test Results
TP-1	21

APPENDIX B

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RECOMMENDED GRADING SPECIFICATIONS

FOR

FRANKLIN RIDGE ROAD EXTENSION
SERRA MESA COMMUNITY
SAN DIEGO, CALIFORNIA

PROJECT NO. G1535-52-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon Incorporated. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, adverse weather, result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition

4. CLEARING AND PREPARING AREAS TO BE FILLED

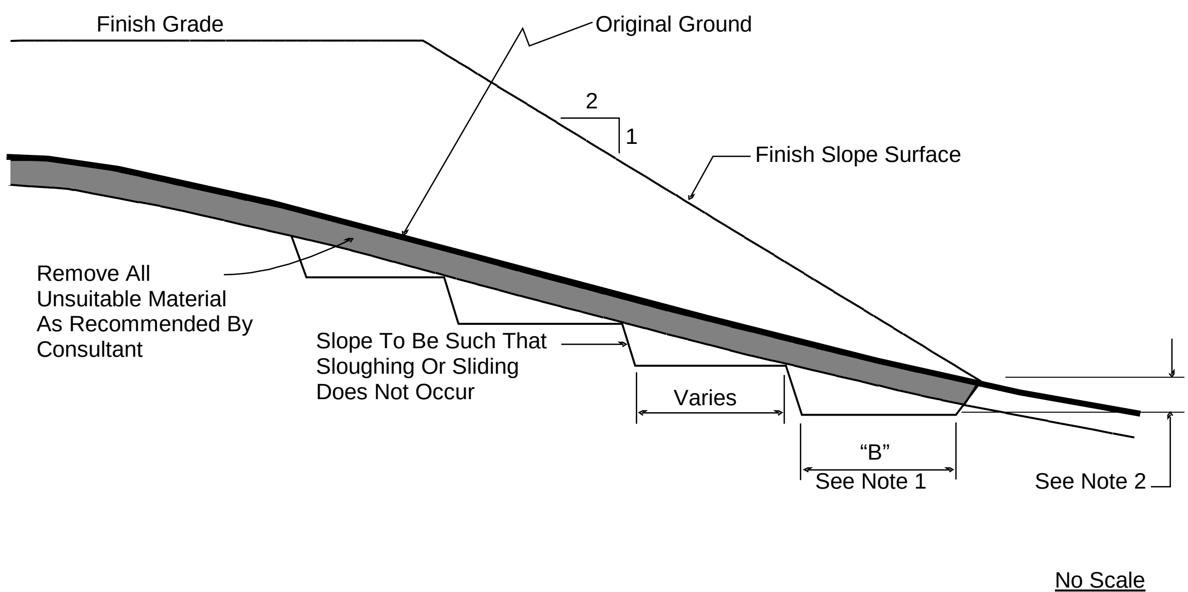
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Any asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557-09.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557-09. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196-09, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of “passes” have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. OBSERVATION AND TESTING

7.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

7.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

7.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

7.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

7.5 The Consultant should observe the placement of subdrains, to verify that the drainage devices have been placed and constructed in substantial conformance with project specifications.

7.6 Testing procedures shall conform to the following Standards as appropriate:

7.6.1 Soil and Soil-Rock Fills:

7.6.1.1 Field Density Test, ASTM D 1556-07, *Density of Soil In-Place By the Sand-Cone Method.*

7.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938-08A, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

7.6.1.3 Laboratory Compaction Test, ASTM D 1557-09, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop.*

7.6.1.4. Expansion Index Test, ASTM D 4829-08A, *Expansion Index Test*.

7.6.2 Rock Fills

7.6.2.1 Field Plate Bearing Test, ASTM D 1196-09 (Reapproved 1997) *Standard Method for Nonreparative Static Plate Load Tests of Soils and Flexible Pavement Components, For Use in Evaluation and Design of Airport and Highway Pavements.*

8. PROTECTION OF WORK

8.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

8.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

9. CERTIFICATIONS AND FINAL REPORTS

9.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

9.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.

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