UPDATE GEOTECHNICAL INVESTIGATION

MULTI-FAMILY DEVELOPMENT
BUNDY CANYON ROAD
NEAR TULIP LANE
APN 367-250-008
WILDOMAR, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

DARLING DEVELOPMENT GROUP
WESTLAKE VILLAGE, CALIFORNIA

FEBRUARY 25, 2016
PROJECT NO. T2145-22-02

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. T2145-22-02
February 25, 2016

Darling Development Group
PO Box 6579
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Attention: Mr. Dick Darling

Subject: UPDATE GEOTECHNICAL INVESTIGATION
MULTI-FAMILY DEVELOPMENT
BUNDY CANYON ROAD AT TULIP LANE
APN 367-250-008
WILDOMAR, CALIFORNIA

Dear Mr. Darling:

In accordance with the request of Mr. Grant Becklund and your authorization of Proposal IE-1520 dated October 15, 2015, Geocon West, Inc. (Geocon) herein submits the results of our update geotechnical investigation and infiltration testing for the proposed multi-family development located at Bundy Canyon Road near Tulip Lane, in the city of Wildomar, California. The accompanying report presents our findings, conclusions and recommendations pertaining to the geotechnical aspects of the proposed development. Based on the results of this study, it is our opinion the site is considered suitable for the proposed development provided the recommendations of this report are followed.

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 WEST, INC.

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UPDATE GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our update geotechnical investigation and infiltration testing for the proposed multi-family development located south of Bundy Canyon Road, near Tulip Lane, in the city of Wildomar, California as depicted on the *Vicinity Map*, Figure 1. The purpose of the investigation is to evaluate subsurface soil and geologic conditions at the site and, based on the conditions encountered; provide recommendations pertaining to the geotechnical aspects of developing the property to accommodate the proposed multi-family residential housing development. We utilized the referenced *Preliminary Grading Plan*, prepared by Grant Becklund Civil Engineering as the base map for our *Geotechnical Map*, Figure 2.

The scope of our investigation included review of aerial photographs and available geotechnical reports in the vicinity of the site, geologic mapping, subsurface exploration, infiltration testing, laboratory testing, engineering analyses, and the preparation of this report. A summary of the information reviewed for this study is presented in the *List of References.*

Our field investigation for the site included the excavation of eight test pits and two infiltration tests. *Appendix A* presents a discussion of the field investigation, logs of the excavations, and infiltration test data. The approximate locations of the exploratory excavations are presented on the *Geotechnical Map* (Figure 2). We performed laboratory tests on soil samples obtained from the exploratory excavations to evaluate pertinent physical and chemical properties for engineering analysis. The results of the laboratory testing are presented in *Appendix B.* Logs of air track borings, test pits, and laboratory testing from previous investigations are provided in *Appendix C*. The approximate locations of previous exploratory excavations are shown on Figure 2.

We referred to the *Plot Plan* and *Preliminary Grading Plan* prepared by Grant Becklund Engineering for site development plan and preliminary grading information. References to elevations presented in this report are based on the referenced topographic information. Geocon does not practice in the field of land surveying and is not responsible for the accuracy of such topographic information.

2. SITE AND PROJECT DESCRIPTION

Site development is proposed to consist of 140 dwelling units in six buildings that will be two to three stories in height. Additional construction will include a single story recreation building pool area, basketball court, picnic area, and associated infrastructure improvements. The site is approximately 28.3 acres and is located south of Bundy Canyon Road, near Tulip Lane, and east of the eastern terminus of Windwood Lane, in the city of Wildomar.

Topography within the site is comprised of granitic bedrock peaks along an arcuate ridgeline in the center of the site with a northwest trending ridgeline in the southeast corner of the site and a lesser peak near the southwest corner of the site. Alluvium-filled valleys are present along the southeastern and southwestern portion of the site. An oak lined ephemeral stream drains to west along the northern border of the site. Vegetation on the remainder of the property consists predominately of moderate to thick brush. A few trails and remnant dirt roads from a previous investigation were observed during our exploration. Drainage in the northern half of the site is to the north into the west flowing ephemeral stream. Drainage on the southern half is predominately to the west to an existing drainage culvert associated with an adjacent neighborhood. A small amount of drainage in the southeastern portion of the site is to the east towards a low lying area on the adjacent property. Site elevations range from a low of approximately 1,450 feet above mean sea level (MSL) in the in the stream bed in the northwest portion of the site to a high of approximately 1,685 feet MSL in the east central portion of the site.

The site will be graded to create level building pads to accommodate the development. Maximum cuts of approximately 95 feet are planned near proposed Building 2. Maximum fills of approximately 75 feet are planned in the southwest corner of the site. Preliminary evaluation has determined that the site will be balanced, with approximately 430,000 cubic yards each of cut and fill

We anticipate the residential buildings will be of typical wood or light metal frame construction and will be founded on conventional shallow foundations with concrete slabs-on-grade or post-tensioned foundation systems. Infiltration basins/structures are preliminarily proposed in the southwest corner of the site. Primary access to the site will be gained off of Bundy Canyon Road, with a secondary access off of the existing Windwood Lane.

3. GEOLOGIC SETTING

The site is located within the Perris Block of the Peninsular Ranges Geomorphic Province. The Perris Block is characterized by granitic highlands which display multiple elevated erosional surfaces surrounded by alluviated valleys. The Peninsular Ranges are bound by the Transverse Ranges (San Gabrielle and San Bernardino Mountains) to the north, the Colorado Desert Geomorphic Province to the east. The Province extends westward into the Pacific Ocean and southward to the tip of Baja California. Overall the Province is characterized by Cretaceous-age granitic rock and a lesser amount of Mesozoic-age metamorphic rock overlain buy terrestrial and marine sediments. Faulting within the province is typically northwest trending and includes the San Andreas, San Jacinto, Elsinore, and Newport-Inglewood faults. Locally, the site is just east of the southern portion of Elsinore Valley, east of the Glenn Ivy segment of the Elsinore fault zone. Cretaceous-age granitic rocks associated with the Paloma Valley Ring Complex compose the majority of the site, with lesser amounts alluvium on the fringes.

4. GEOLOGIC MATERIALS

4.1 General

Site geologic materials encountered consist of previously placed artificial fill, alluvium, and granitic bedrock. Previously placed artificial fill was encountered along the western edge of the site, associated with the neighboring housing tract. Alluvium is present within the ephemeral stream along the north of the site, and in the southwest and southeast portions of the site. Cretaceous-age granitic bedrock (granodiorite) makes up the remainder of the site and underlies the site at depth. The lateral extent of the materials encountered is shown on the *Geotechnical Map* (Figure 2). The descriptions of the soil and geologic conditions are shown on the excavation logs located in *Appendix A* and described herein in order of increasing age.

4.2 Previously Placed Artificial Fill - (Qpf)

Although not encountered in the explorations, previously placed artificial fill was observed along the western portion of the site, in association with the grading of the neighboring housing tract. The fill appears to be locally derived silty sand which was appeared to be dry to slightly moist and medium dense to dense. Geocon has not been provided documentation pertaining to the quality of the fill. Therefore, this fill should be evaluated during grading operations and, if necessary replaced with documented fill in conformance with the recommendations herein.

4.3 Topsoil - (No Map Symbol)

Topsoil was encountered in test pits GT-1 through Gt-3, and GT-8 to depths of one to three feet overlying alluvium and granitic bedrock. Topsoil consisted of silty, fine to coarse sand that was loose, slightly moist, brown, and contained some gravel. Due to loose consistency of the topsoil, complete removal and replacement with compacted fill is recommended.

4.4 Alluvium - (Qal)

Alluvium was encountered within test pits GT-2 through GT-8 during this investigation within the southwest and southeast drainages. We encountered alluvium deeper than 15 feet, beyond the reach of our excavation equipment. The alluvium was encountered to depths of 10 ½ feet within the southeastern drainage. The alluvium along the stream to the north was not evaluated during this investigation. However a previous investigation indicated depths of five to 15 feet on the parcel immediately to the east. Similar depths can be anticipated on this site. Alluvium, as encountered, consisted of silty sand with varying amounts of gravel which was loose to medium dense. Although some of the alluvium is medium dense, we anticipate the consistency, density, and moisture content to be variable and are therefore, recommending complete removal of the alluvial soils and replacement with compacted fill.

4.5 Granitic Bedrock (Granodiorite) - (Kpvg)

Cretaceous-age granitic bedrock consisting predominately of granodiorite with lesser amounts of monzogranite dominates the site and underlies the alluvium. As observed and based on our previous investigation at the site, the granitic bedrock is generally moderately weathered, dense, and rippable to marginally rippable. The upper approximately five feet of the granitic rock is highly weathered and excavates as silty fine to coarse sand. Some zones of non-rippable rock were observed in the air track borings and should be expected to be encountered during grading (see *Appendix C*). Even though the granitic rock is moderately fractured, oversize material (greater than 12 inches) should be expected to be generated during grading. Oversize material that is generated during grading should be placed in fills deeper than 10 feet below finish grade, and at least one foot below deepest utilities if placed within a utility corridor. The granitic rock is suitable to support fill or proposed structures.

5. GEOLOGIC STRUCTURE

The geologic structure consists of generally moderately weathered and jointed granitic bedrock overlain by surficial soils.

6. GROUNDWATER

Groundwater was not encountered during our investigation and is not anticipated to be encountered during grading. However, dependent upon the grading schedule and regional weather conditions, perched water may be encountered within the alluvium near the bedrock contact, and in fractures within the granitic bedrock after a rain event. If water is encountered during construction, it is our opinion that the use of sump pumps, diversion and/or damming techniques could be used to manage nuisance water encountered.

7. GEOLOGIC HAZARDS

7.1 Faulting

The site, like the rest of southern California, is located within a seismically active region near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional faults such as the San Andreas, San Jacinto and Elsinore fault zones. These fault systems are estimated to produce up to approximately 55 millimeters of slip per year between the plates (Harden, 1998).

The site is not located within a State of California "Alquist-Priolo Earthquake Fault Zone" for fault rupture hazard (CGS 2015), nor is the site located within a Riverside County Fault Zone (Riverside GIS, 2016). There are several major late Quaternary active/potentially active faults that are within a 100-kilometer radius of the site (Blake, 2000). The nearest known active fault to the site is the Glen Ivy section of Elsinore fault zone located approximately 2,300 feet west of the project site. The Elsinore

fault zone is a right-lateral fault, with multiple *en echelon* faults which create tensional basins like that of Lake Elsinore. The Elsinore fault zone is capable of producing an earthquake with an estimated maximum moment magnitude of M_W 7.5, and has an associated slip-rate of 1.0 mm/year (CDMG, 1996).

7.2 Seismic Design Criteria

We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 7.2.1 summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The building structure and improvements should be designed using a Site Class D. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2013 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 7.2.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 7.2.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.265g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.907g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.265g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	1.361g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.510g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.907g	Section 1613.3.4 (Eqn 16-40)

Table 7.2.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 7.2.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.894	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.894g	Section 11.8.3 (Eqn 11.8-1)

Conformance to the criteria in Tables 7.2.1 and 7.2.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.3 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless/silt or clay with low plasticity, static groundwater is encountered within 50 feet of the surface, and soil relative densities are less than about 70 percent. If the four previous criteria are met, a seismic event could result in a rapid pore-water pressure increase from the earthquake-generated ground accelerations. Seismically induced settlement may occur whether the potential for liquefaction exists or not. Based on the dense to very dense nature of the granitic bedrock, the dense nature of proposed fill placed at the site, and the lack of groundwater at the site, liquefaction and seismically induced settlement of soil is not a design consideration.

7.4 Expansive Soil

The geologic units generally consisted of silty sands to gravelly sands which are expected to have a low expansion potential. Previous laboratory testing results (*Appendix C*) indicate a sample of the fine-grained soil units exhibit an expansion index of 25 which would classify as “low” in accordance with ASTM International (ASTM) D4829. If medium to highly expansive soils are encountered during grading, they should be kept at least four feet below proposed structural, flatwork, or paving improvements.

7.5 Collapsible Soils

Alluvial soils may exhibit some degree of collapse potential when loaded to the anticipated post-grading pressures. Remedial grading (removal of alluvium) should be performed to mitigate the effects of the collapsible soils.

7.6 Landslides

The property is in an area of moderate relief with intervening alluviated valleys. No landslides were mapped on the references reviewed nor were landslides observed during our field investigation. Due to the nature of the material (granitic bedrock) that makes up the slopes, and the proposed grading, the likelihood of landsliding at the site is not a design consideration. We did not observe other evidence of slope stability issues on or directed toward the site during our aerial photograph review or our field investigation.

7.7 Rock Fall Hazards

The hills on and adjacent to the site consist of granitic bedrock. A few boulders were observed during our field work. However, the proposed grading will mitigate any potential for rockfall affecting the site. The natural slopes below the project area are relatively free of boulders, covered with brush, and topsoil which will mitigate rock hall hazard. Therefore, rock fall issues are not a design consideration for this project.

7.8 Slope Stability

We anticipate proposed grading at the project site will include fill slopes with maximum heights of approximately 75 feet , with maximum inclinations of 2:1 (h:v), and horizontal benches or roadways at about mid-height within the slopes. Minor cut slopes into the granitic bedrock with maximum heights of approximately 15 feet and maximum inclinations of 2:1 (h:v) are anticipated in cut areas of the site. In general, it is our opinion that cut slopes in the granitic bedrock and fill slopes constructed with on-site soils as described above will possess Factors of Safety of 1.5 or greater under static conditions and 1.1 or greater under seismic conditions. Slope stability analyses are presented on Figures 3 through 8. Additional slope stability analyses should be performed as needed during further development of the grading plans. Cut slopes should be geologically mapped during grading. Fill keys should be constructed in accordance with the standard grading specifications in *Appendix D*. Grading of cut and fill slopes should be designed in accordance with the requirements of the local building codes of the City of Wildomar and the 2013 California Building Code (CBC).

7.9 Tsunamis and Seiches

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). The site is located 24 miles from the nearest coastline; therefore, the negligible risk associated with tsunamis is not a design consideration.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. Canyon Lake is approximately 3½ miles north of the site and Lake Elsinore is approximately 4½ miles northwest of the site. Both lakes are at lower elevations (100 and 300 feet lower) than the project site. Therefore, seiches are not a design consideration for the site.

8.0 SITE INFILTRATION

Percolation testing was performed in accordance with Table 1 Infiltration Basin Option 2 of Appendix A of Riverside County – Low Impact Development BMP Design Handbook (Handbook). The percolation tests were run in accordance with Section 2.3 Deep Percolation Test Method. This method requires two percolation tests and one deep (extending 10 feet below percolation test elevation) excavation per basin.

The test pit and percolation test locations are depicted on the *Geotechnical Map,* Figure 2. Test pit logs and percolation test data are presented in *Appendix A.* Descriptions of the testing procedures, and test results are provided below.

A storm water infiltration structure is planned for the southwest corner of the site at approximately 1 foot below existing grade. Geocon hand excavated two percolation test holes (P-1 and P-2) to depths of 1 foot below grade. Soils encountered within the excavations consisted of predominately medium dense silty sand. No groundwater was observed within the excavations. A 12-inch diameter hole was hand excavated and lined with a perforated 5-gallon bucket. Approximately two inches of gravel was placed at the bottom of the hole. The test locations were pre-saturated with five gallons of water. The percolation testing began approximately 24 hours after the holes were pre-saturated. Percolation data sheets are presented in *Appendix A* of this report. Calculations to convert the percolation test rate to infiltration test rate in accordance with Section 2.3 of the Handbook are presented in Table 8.0. Please note that the Handbook requires a factor of safety of 3 be applied to the values below based on the test method used.

TABLE 8.0
INFILTRATION TEST RATES

	P-1	P-2
Soil Type	Normal	Normal
Change in head over time:ΔH	0.5 in	0.7 in
Time Interval (minutes): Δt	30 min	30 min
Radius of test hole: r	6 in	6 in
Average head over time interval: Havg	10.2 in	10.1 in
Tested Infiltration Rate: It	0.2 in/hr	0.3 in/hr

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 General

9.1.1 It is our opinion that soil or geologic conditions were not encountered during the investigation that would preclude the proposed development of the project provided the recommendations presented herein are followed and implemented during construction.

9.1.2 Potential geologic hazards at the site include seismic shaking, highly erodible soils, and compressible near surface soils.

9.1.3 Based on our review of aerial photographs and readily available geotechnical reports, we do not believe that faulting is present on the site.

9.1.4 Previously placed fill associated with the neighboring development along the southwest boundary should be evaluated during to grading for its suitability to support new documented fill.

9.1.5 The topsoil, alluvium, and highly weathered granitic bedrock are considered unsuitable for the support of compacted fill or settlement-sensitive improvements based on the potential compressibility of the units. Remedial grading of the surficial soil will be required as discussed herein. Over excavation of cut fill transition building pads will be required, or alternatively in cut fill transition building pads, deepened foundations may be used to support the structures entirely in the granitic bedrock. New documented fill is considered suitable to support additional fill and the proposed structures and improvements.

9.1.6 We did not encounter groundwater during our investigation. Seepage and perched groundwater conditions may be encountered during the grading operations, particularly during the rainy seasons.

9.1.7 Subdrains will be required in areas where fill is placed over bedrock such as keyways or in canyons. *Appendix D* provides general subdrain recommendations. If fill has to be placed below a canyon subdrain in order to achieve proper flow, that fill should be compacted to 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. The approximate locations of anticipated subdrains are shown on Figure 2. The actual location of subdrains should be determined by Geocon during grading.

9.1.8 In general, slopes should possess calculated factors of safety of at least 1.5 in static conditions and 1.1 in seismic conditions with slopes inclined as steep as 2:1 (h:v), maximum heights of 75 feet, and horizontal benches or roadways as shown on the referenced *Preliminary Grading Plan*. Slopes should be reevaluated once final grading plans have been prepared for the site.

9.1.9 If cut slopes expose adverse geologic conditions, stabilization fills will likely be required.

9.1.10 Proper drainage should be maintained in order to preserve the engineering properties of the fill in the sheet-graded pads and slope areas and reduce the potential for erosion of the granular soils. Recommendations for site drainage are provided herein.

9.1.11 The majority of the granitic rock should be considered rippable based on previous air track borings (see *Appendix C*). However, non-rippable zones and corestones (hard intact boulders) should be expected to be encountered. We understand that quarry operations north of the site by Buck Kemmis encountered excavatable bedrock with isolated core stones and blasting was not required at the quarry. Blasting or other excavation techniques may be required in these areas.

9.1.12 Oversize material generated should be able to be used as fill provided the recommendations provided in *Section 9.3* and *Appendix D* are followed.

9.1.13 Fill placed in deep fill areas (greater than 50 feet in thickness), should be compacted to 95 percent relative compaction and settlement monitors should be installed and monitored to measure settlement of the newly placed fill. Construction of settlement sensitive structures should be delayed until settlements are within tolerable levels.

9.2 Soil Characteristics

9.2.1 The site soils soil encountered in the field investigation are considered to be “expansive” (Expansion Index [EI] greater than 20) as defined by 2013 California Building Code (CBC) Section 1803.5.3. The laboratory test results would classify the soil as having a “low” expansion index in accordance with ASTM D4829. Table 9.2.1 presents soil classifications based on the EI.

TABLE 9.2.1
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2013 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

9.2.2 Based on the material classifications and laboratory testing, fine grained site soils generally possess a very low to low expansion potential (EI between 0 and 50). Medium to highly expansive soils, if encountered, should not be placed within four feet of the proposed foundations, flatwork or paving improvements. Additional testing for expansion potential should be performed once final grades are achieved.

9.2.3 Laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests indicate that the on-site materials at the location tested possess a sulfate content of 0.001% equating to an exposure class of S0 (Negligible) to concrete structures as defined by 2013 CBC Section 1904.3 and ACI 318. Table 9.2.3 presents a summary of concrete requirements set forth by 2013 CBC Section 1904.3 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 9.2.3
REQUIREMENTS FOR CONCRETE
EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Exposure	Exposure Class	Water-Soluble Sulfate Percent by Weight	Cement Type	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Not Applicable	S0	0.00-0.10	--	--	2,500
Moderate	S1	0.10-0.20	II	0.50	4,000
Severe	S2	0.20-2.00	V	0.45	4,500
Very Severe	S3	> 2.00	V+ Pozzolan or Slag	0.45	4,500

9.2.4 Laboratory testing indicates the site soils have a pH of 6.4, and possess 86 parts per million (ppm) chloride, and have a minimum resistivity of 3,800 ohm-cm. Based on the lab test results, the site soils would not be classified as "corrosive" to metal improvements, in accordance with the Caltrans Corrosion Guidelines (Caltrans, 2012).

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

9.3 Grading

9.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in *Appendix D* and the Grading Ordinances of the City of Wildomar.

9.3.2 Prior to commencing grading, a preconstruction conference should be held at the site with the city inspector, 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.

9.3.3 Site preparation should begin with the removal of previous structures and infrastructure, deleterious material, debris, buried trash, and vegetation. The depth of removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.

9.3.4 Topsoil, alluvium, and highly weathered granitic bedrock within a 1:1 (h:v) projection the limits of grading should be removed to expose competent granitic bedrock. The area of previously placed fill along the western side of the site will be observed and evaluated during grading. The actual depth of removal should be evaluated by the engineering geologist during grading operations. We expect that removals will need to extend laterally beyond the outside edge of building or toes of slopes at a 1:1 (h:v) projection. The bottom of the excavations should be scarified to a depth of at least 1 foot, moisture conditioned as necessary to near to slightly above optimum moisture content, and properly compacted.

9.3.5 Bedrock exposed at finish grade (Buildings 1, 2, and 3) and in cut fill transitions (Building 4 and Rec Building), and areas within proposed structural areas should be over excavated to remove the differential support conditions. Over excavations should extend a minimum of three feet below pad grade or H/3 (where H is deepest fill in building envelope area), whichever is greater. Over excavations should be sloped so a bath-tub like geometry does not result from the over excavation. In cut fill transition areas where overexcavation is not

practical due to non-rippable rock being encountered beneath the building pad elevation, the buildings may be supported on deepened foundations extending through the fill and into the underlying bedrock.

9.3.6 Consideration should be given to overexcavation of granitic bedrock and replacement with compacted fill in areas of utility corridors, flatwork, parking lots, driveways, and any other improvements that would extend into granitic bedrock in order to facilitate excavation for construction of these improvements.

9.3.7 We should observe the removal bottoms to verify suitable material is encountered. Deeper excavations may be required if dry, loose, or soft materials are present at the base of the removals.

9.3.8 The fill placed within 4 feet of proposed foundations should possess a “low” expansion potential (EI of 50 or less).

9.3.9 If perched groundwater or saturated materials are encountered during remedial grading, extensive drying and mixing with dryer soil will be required. The excavated materials should then be moisture conditioned as necessary to near to slightly above optimum moisture content prior to placement as compacted fill.

9.3.10 The site soils and excavated granitic bedrock are suitable for re-use as engineered fill provide they are free of organics, and deleterious material. Material greater than 6 inches should not be placed within 10 feet of finish grade.

9.3.11 The site should be brought to finish grade elevations with fill compacted in layers. Layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be 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. Fills deeper than 50 feet should be compacted to 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

9.3.12 Import fill (if necessary) should consist of granular materials with a “low” expansion potential (EI of 50 or less), generally free of deleterious material and rock fragments larger than 6 inches, noncorrosive, and should be compacted as recommended herein. Geocon 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.

9.3.13 Oversize material (rock bigger than 12-inches) should be expected to be generated during grading operations with the granitic bedrock. Oversize rock can be used as fill as long as it is placed in accordance with City of Wildomar requirements, Riverside County Grading Ordinance and the recommendations in *Appendix D* of this report. The fill portion of the site should be able to accommodate oversize material as long as it is kept at least ten feet below finish grade, one foot below deepest utility in all utility corridors, and at least 15 feet horizontally from the face of slope.

9.3.14 Rock blanket fills are allowable provided they are placed in accordance with rock placement details provided in *Appendix D*.

9.3.15 Rocks of up to four feet in diameter may by placed individually or in windrows. The placement of individual rocks or windrows should allow compaction equipment can traverse on either side. Granular soil should be used to fill the void spaces around and beneath the rocks, and should be flooded.

9.3.16 Rock windrows should be placed parallel to each other, have a minimum horizontal spacing of 12 feet center-to-center, have a five foot horizontal offset from the underlying course, and a two foot vertical offset from the windrow below and above.

9.3.17 If possible, a subdrain should be placed at the base of a rock fill and outletted to a permanent drainage structure.

9.4 Graded Slopes

9.4.1 Fill slopes should be overbuilt at least 2 feet and cut back to grade. The slopes should be track-walked at the completion of each slope such that the fill is compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content to the face of the finished slope.

9.4.2 Finished slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. In addition, the slopes should be drained and properly maintained to reduce erosion. Water should not be allowed to flow down slopes, construction of earth berms, lined v-ditches or similar are recommended.

9.4.3 Although the proposed slopes are anticipated to have adequate factors of safety, natural factors may result in slope creep and/or lateral fill extension over time. Slope creep is due to alternate wetting and drying of fill soils resulting in downslope movement. Slope creep

occurs throughout the life of the slope and may affect improvements within about 15 feet of the top of slope, depending on the slope height. Slope creep can results in differential settlement of the structures supported by the slope. Lateral fill extension (LFE) occurs when expansive soils within the slope experience deep wetting due to rainfall or irrigation. LFE is mitigated as much as practical during grading by placing expansive soils at slightly greater than optimum moisture content.

9.4.4 Landscaping activities should avoid over steepening of slopes or grade changes along slopes. Backfill of irrigation lines should be compacted to 90 percent of the maximum dry density as evaluated by ASTM D1557. Vegetation should be light weight with variable root depth.

9.4.5 Excessive watering should be avoided; only enough irrigation to support vegetation suitable to the prevailing climate should be applied. Irrigation of natural, ungraded slopes should not be performed. Drainage or irrigation from adjacent improvements should not be directed to the tops of slopes. Drainage should be directed toward streets and approved drainage devices. Areas of seepage may develop after periods of heavy rainfall or irrigation.

9.4.6 Homeowners and maintenance associations should be made aware of the potential for slope creep, LFE, and erosion and be provided with these recommendations on how to reduce the likelihood of its occurrence.

9.5 Earthwork Grading Factors

9.5.1 Estimates of shrinkage factors are based on empirical judgments comparing the material in its existing or natural state as encountered in the exploratory excavations to a compacted state. Variations in natural soil density and in compacted fill density render shrinkage value estimates very approximate. As an example, the contractor can compact the fill to a dry density of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has an approximately 10 percent range of control over the fill volume. Table 9.5.1 below proves estimated shrink/bulk factors.

TABLE 9.5.1
ESTIMATED SHRINK/BULK FACTORS

Soil Unit	Shrink/Bulk Factors
Topsoil	5-15 percent shrink
Alluvium	7-12 percent shrink
Granitic Bedrock	5-15 percent bulk

Please note that this estimate is for preliminary quantity estimates only. Due to the variations in the actual shrinkage/bulking factors, a balance area should be provided to accommodate variations.

9.6 Settlement of Proposed Fill

9.6.1 The post-grading settlement (hydrocompression) could reach up to 1 inch. We expect the settlement will occur over many years depending on the influx of rain and irrigation water into the fill. The settlement will likely be linear from the time the fill is placed to the end of the settlement period depending on the permeability of the fill soil. We do not expect the settlement will impact proposed utilities with gradients of 1 percent or greater. In addition, foundation recommendations are provided herein based on the maximum and differential fill thickness to account for potential fill settlement. Surface settlement monuments should be placed on fills deeper than 50 feet. The settlement monuments should be surveyed periodically to evaluate settlement of the newly placed fill. Construction of settlement sensitive structures should be delayed until expected settlements are within tolerable levels.

9.7 Foundation and Concrete Slabs-On-Grade Recommendations

9.7.1 The foundation recommendations presented herein are for the various proposed buildings. We understand that the buildings will be supported on either conventional shallow foundations with concrete slabs-on-grade or post-tensioned foundation systems.

9.7.2 In cut fill transition areas, the foundations may be founded in compacted fill where the bedrock is overexcavated in accordance with the grading recommendations, or the foundations may be extended such that the building is founded entirely on undisturbed bedrock in order to avoid overexcavation of non-rippable areas.

9.7.3 We separated the foundation recommendations into three categories based on either the maximum and differential fill thickness or Expansion Index. The foundation category criteria for the anticipated conditions are presented in Table 9.7.3. Final foundation categories will be evaluated once site grading has been completed.

TABLE 9.7.3
FOUNDATION CATEGORY CRITERIA

Foundation Category	Maximum Fill Thickness, T (Feet)	Differential Fill Thickness, D (Feet)	Expansion Index (EI)
I	$T<20$	$D<10$	$EI\leq50$
II	$20\leq T<50$	$10\leq D<20$	$50<EI\leq90$
III	$T\geq50$	$D\geq20$	$90<EI\leq130$

9.7.4 Post-tensioned concrete slab and foundation systems may be used for the support of the proposed structures. The post-tensioned systems should be designed by a structural engineer

experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI), as required by the 2013 California Building Code (CBC Section 1808.6). Although this procedure was developed for expansive soil conditions, we understand it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented on Table 9.7.4 for the particular Foundation Category designated. The parameters presented in Table 9.7.4 are based on the guidelines presented in the PTI, Third Edition design manual. The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer.

TABLE 9.7.4
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) Third Edition Design Parameters	Foundation Category		
	I	II	III
Thornthwaite Index	-20	-20	-20
Equilibrium Suction	3.9	3.9	3.9
Edge Lift Moisture Variation Distance, e_M (feet)	5.3	5.1	4.9
Edge Lift, y_M (inches)	0.61	1.10	1.58
Center Lift Moisture Variation Distance, e_M (feet)	9.0	9.0	9.0
Center Lift, y_M (inches)	0.30	0.47	0.66

9.7.5 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). In addition, the membrane should be installed in accordance with manufacturer's recommendations and ASTM requirements and installed in a manner that prevents puncture. The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity-controlled environment.

9.7.6 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. Placement of 3 inches and 4 inches of sand is common practice in Southern California for 5-inch and 4-inch thick slabs, respectively. The foundation engineer should provide appropriate concrete mix design criteria and curing measures that may be utilized to assure proper curing of the slab to reduce the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation engineer present concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

9.7.7 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. A wall/column footing dimension detail is provided on Figure 9. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer.

9.7.8 If the structural engineer proposes a post-tensioned foundation design method other than the 2013 CBC:

- The deflection criteria presented in Table 9.7.3 are still applicable.
- Interior stiffener beams should be used for Foundation Categories II and III.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches, 18 inches and 24 inches for foundation categories I, II, and III, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

9.7.9 Our experience indicates post-tensioned slabs are susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. Because of the placement of the reinforcing tendons in the top of the slab, the resulting eccentricity after tensioning reduces the ability of the system to mitigate edge lift. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

9.7.10 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system.

9.7.11 Foundations in compacted fill may be designed for an allowable soil bearing pressure of 3,500 pounds per square foot (psf) (dead plus live load). This value may be increased by 250 psf for each additional foot in depth and 200 psf for each additional foot of width to a maximum value of 4,500 psf. Foundations extending to the underlying granitic bedrock may be designed for an allowable soil bearing pressure of 5,000 psf. The allowable bearing pressure may be increased by one-third for transient loads due to wind or seismic forces. We estimate the total settlements under the imposed allowable loads to be about 1 inch with differential settlements on the order of ½ inch over a horizontal distance of 40 feet.

9.7.12 As an alternate to post-tensioned foundation systems, conventional shallow foundation with a concrete slab-on-grade may be used for support of the proposed structures. Conventional shallow foundations in compacted fill may be designed for an allowable soil bearing pressure of 3,500 pounds per square foot (psf) (dead plus live load). This value may be increased by 250 psf for each additional foot in depth and 200 psf for each additional foot of width to a maximum value of 4,500 psf. Foundations extending to the underlying granitic bedrock may be designed for an allowable soil bearing pressure of 5,000 psf. The allowable bearing pressure may be increased by one-third for transient loads due to wind or seismic forces. We estimate the total settlements under the imposed allowable loads to be about 1 inch with differential settlements on the order of ½ inch over a horizontal distance of 40 feet. Table 9.7.12 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 9.7.12
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY

Foundation Category	Minimum Footing Embedment Depth (inches)	Continuous Footing Reinforcement	Interior Slab Reinforcement
I	12	Two No. 4 bars, one top and one bottom	6 x 6 – 10/10 welded wire mesh at slab mid-point
II	18	Four No. 4 bars, two top and two bottom	No. 3 bars at 24 inches on center, both directions at slab mid-point
III	24	Four No. 5 bars, two top and two bottom	No. 3 bars at 18 inches on center, both directions at slab mid-point

9.7.13 The embedment depths presented in Table 9.7.12 should be measured from the lowest adjacent pad grade for both interior and exterior footings. The conventional foundations should have a minimum width of 12 inches and 24 inches for continuous and isolated footings, respectively. Figure 9 presents a wall/column footing dimension detail depicting lowest adjacent pad grade.

9.7.14 Isolated footings, if present, should have the minimum embedment depth and width recommended for conventional foundations for a particular foundation category. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended for Category III. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams.

9.7.15 Foundations in cut fill transition where the bedrock is non-rippable should be extended to the underlying granitic bedrock in the fill areas to avoid differential support conditions of the building. Foundations should be embedded at least 6 inches into the granitic bedrock, and at least 12 inches beneath lowest adjacent pad grade. Where the granitic bedrock in the fill areas is deeper than practical for deepening of the foundations, such as the southern portion of Building 4, Geocon should be contacted for additional recommendations if the bedrock is non-rippable. Alternative foundations may include using cast-in-drilled-hole piles that bear on the granitic bedrock in combination with a structural floor.

9.7.16 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in newly compacted fill.

9.7.17 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineered fill may be computed as an equivalent fluid having a density of 350 pounds per cubic foot with a maximum earth pressure of 3,500 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

9.7.18 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in such concrete placement.

9.7.19 Where buildings or other improvements are planned near the top of a slope steeper than 3:1 (horizontal to vertical), special foundations and/or design considerations are recommended due to the tendency for lateral soil movement to occur.

- For fill slopes less than 20 feet high or cut slopes regardless of height, footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, building foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but not to exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope.
- Geocon should be contacted to review the pool plans and the specific site conditions to provide additional recommendations, if necessary.
- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support.
- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil movement without causing extensive distress. Geocon should be consulted for specific recommendations.

9.7.20 The recommendations of this report are intended to reduce the potential for cracking of slabs and foundations due to expansive soil (if present) and differential settlement of fill soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

9.7.21 Geocon should be consulted to provide additional design parameters as required by the structural engineer.

9.8 Exterior Concrete Flatwork

9.8.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein assuming the subgrade materials possess an Expansion Index of 50 or less. Subgrade soils should be compacted to 90 percent relative compaction. Slab panels should be a minimum of 4 inches thick and when in excess of 8 feet square should be reinforced with 6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh or No. 3 reinforcing bars spaced 18 inches center-to-center in both directions to reduce the potential for cracking. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

9.8.2 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade or differential settlement. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork.

9.8.3 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

9.8.4 The recommendations presented herein are intended to reduce the potential for cracking of exterior slabs as a result of differential movement. However, even with the incorporation of the recommendations presented herein, slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

9.9 Conventional Retaining Walls

9.9.1 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 35 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2:1 (horizontal to vertical), an active soil pressure of 60 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an EI of 50 or less. For those walls where backfill materials do not conform to the criteria herein, Geocon should be consulted for additional recommendations.

9.9.2 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, an additional fluid density of 25 pcf should be added to the active soil pressure.

9.9.3 The structural engineer should determine the seismic design category for the project. If the project possesses a seismic design category of D, E, or F, the proposed retaining walls should be designed with seismic lateral pressure added to the active pressure. The seismic load exerted on the wall should be a triangular distribution with a pressure of 24H (where H is the height of the wall, in feet, resulting in pounds per square foot [psf]) exerted at the bottom of the wall and zero at the top of the wall. We used a peak site acceleration of 0.894g calculated from the 2013 California Building Code and applying a pseudo-static coefficient of 0.33.

9.9.4 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

9.9.5 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140 (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted backfill

(EI of 50 or less) with no hydrostatic forces or imposed surcharge load. Figure 10 presents a typical retaining wall drainage detail. If conditions different than those described are expected or if specific drainage details are desired, Geocon should be contacted for additional recommendations.

9.9.6 In general, wall foundations having a minimum depth and width of 1.5 feet may be designed for an allowable soil bearing pressure of 3,500 psf. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, Geocon should be consulted where such a condition is expected.

9.9.7 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 12 feet. In the event that walls higher than 12 feet or other types of walls are planned, Geocon should be consulted for additional recommendations.

9.10 Lateral Loading

9.10.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid weight of 350 pounds per cubic foot (pcf) should be used for the design of footings or shear keys poured neat against formational materials. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

9.10.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.35 should be used for design.

9.11 Swimming Pool/Spa

9.11.1 If swimming pools or spas are planned, the proposed swimming pool shell bottom should be designed as a free-standing structure and may derive support in newly placed engineered fill or the granitic bedrock. It is recommended that uniformity be maintained beneath the proposed swimming pools where possible, so overexcavation of the bedrock and replacement with compacted fill may be necessary. Geocon should be contacted for additional recommendations where swimming pools are planned over a cut-fill transition.

9.11.2 Swimming pool foundations and walls may be designed in accordance with the *Foundation* and *Retaining Wall* sections of this report (See Sections 9.7 and 9.9). A hydrostatic relief valve should be considered as part of the swimming pool design unless a gravity drain system can be placed beneath the pool shell.

9.11.3 If a spa is proposed it should be constructed independent of the swimming pool and must not be cantilevered from the swimming pool shell.

9.11.4 If the proposed pool is in proximity to the proposed structure, consideration should be given to construction sequence. If the proposed pool is constructed after building foundation construction, the excavation required for pool construction could remove a component of lateral support from the foundations and would therefore require shoring. Once information regarding the pool location and depth becomes available, this information should be provided to Geocon for review and possible revision of these recommendations.

9.12 Preliminary Pavement Recommendations

9.12.1 The final pavement sections for roadways should be based on the R-Value of the subgrade soils encountered at final subgrade elevation. Streets should be designed in accordance with the City of Wildomar specifications (Riverside County standards) when final Traffic Indices and R-Value test results of subgrade soil are completed. Based on the soil types encountered during our investigation an R-Value of 24 for the subgrade soil and an assumed 78 for aggregate base materials have been used for the purposes of this preliminary analysis. Preliminary flexible pavement sections are presented in Table 9.12.1 for road classifications in accordance with the County of Riverside *Roadway Design Requirements* (Standard No. 114).

TABLE 9.12.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Crushed Aggregate Base (inches)
Light-Duty Vehicles and Parking Areas	5.5	24	3.5	7.0
Collector	7.0	24	4.5	10.0
Heavy Truck Vehicles	8.0	24	5.5	12.0

9.12.2 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 near to slightly above optimum moisture content beneath pavement sections.

9.12.3 The crushed aggregated base and asphalt concrete materials should conform to Section 200-2.2 and Section 203-6, respectively, of the *Standard Specifications for Public Works Construction* (Greenbook) and the latest edition of the *County of Riverside Standard*

Specifications. Base materials 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. Asphalt concrete should be compacted to a density of 95 percent of the laboratory Hveem density in accordance with ASTM D 1561.

9.12.4 A rigid Portland cement concrete (PCC) pavement section should be placed in driveway aprons and cross gutters and where desired to support heavy vehicle loads. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R, *Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 9.12.4.

TABLE 9.12.4
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	550 psi
Traffic Category, TC	C and D
Average daily truck traffic, ADTT	100 and 700

9.12.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 9.12.5.

TABLE 9.12.5
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Roadways (TC=C)	6.5
Bus Stops and Truck Parking Areas (TC=D)	8.0

9.12.6 The PCC pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,500 psi (pounds per square inch). Base material will not be required beneath concrete improvements.

9.12.7 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, and taper back to the

recommended slab thickness 4 feet behind the face of the slab (e.g., a 7-inch-thick slab would have a 9-inch-thick edge). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

9.12.8 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 15 feet for the 7-inch-thick or greater slabs (e.g., a 9-inch-thick slab would have a 15-foot spacing pattern). The depth of the crack-control joints and need for sealing of the joints should be determined by the referenced ACI report.

9.12.9 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab. As an alternative to the butt-type construction joint, dowelling can be used between construction joints for pavements of 7 inches or thicker as discussed in the referenced ACI guide.

9.12.10 Where brick or concrete pavers are used, the following preliminary pavement sections may be used. Final pavement sections should be based on the R-value of the soils placed at the roadway subgrade elevation and the type of paver chosen for the pavement surface. Traffic indices have been assumed for walkways, light duty pavements and parking lots, and Collector roads. If the anticipated traffic is different than what we assumed, Geocon should be contacted to provide revised recommendations.

9.12.11 Pavement thicknesses were determined following procedures outlined in the Caltrans California Highway Design Manual. The pavers provide a wearing surface with a similar structural value as asphalt concrete, and additional support for the pavers is provided with the underlying aggregate base. The recommended thickness of aggregate base beneath the pavers is provided in Table 9.12.11.

TABLE 9.12.11
PRELIMINARY PAVER RECOMMENDATIONS

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Recommended thickness of Crushed Aggregate Base (inches)
Walkways	4.0	24	6.0
Light-Duty Vehicles and Parking Areas	5.5	24	8.0
Collector	7.0	24	13.0

9.12.12 Pavers should be installed in accordance with the manufacturer's recommendations, including the use of bedding sand and a geotextile fabric above the aggregate base. The crushed aggregate base should conform to Section 200-2.2 of the Standard Specifications for Public Works Construction (Greenbook). Base materials should be compacted to 95 percent of the laboratory maximum dry density at near to slightly above optimum moisture content as evaluated by ASTM D1557.

9.12.13 The performance of pavement is highly dependent on providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement surfaces will likely result in pavement distress and subgrade failure. Drainage from landscaped areas should be directed to controlled drainage structures. Landscape areas adjacent to the edge of asphalt pavements are not recommended due to the potential for surface or irrigation water to infiltrate the underlying permeable aggregate base and cause distress. Where such a condition cannot be avoided, consideration should be given to incorporating measures that will significantly reduce the potential for subsurface water migration into the aggregate base. If planter islands are planned, the perimeter curb should extend at least 6 inches below the level of the base materials.

9.13 Site Drainage and Moisture Protection

9.13.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2013 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. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

9.13.2 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.

9.13.3 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.

9.13.4 We understand the property may incorporate storm water management devices that promote water infiltration. 9.13.5 If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. Based on our experience with similar clayey soil conditions, infiltration areas are considered infeasible due to the poor percolation and lateral migration characteristics. We have not performed a hydrogeology study at the site. Down-gradient and adjacent structures may be subjected to seeps, movement of foundations and slabs, or other impacts as a result of water infiltration.

9.14 Plan Review

9.14.1 Geocon should review the grading and structural foundation plans for the project prior to final submittal. Additional analyses may be required after review of the project plans.

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 should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous materials was not part of the scope of services provided by Geocon.

2. This report is issued with the understanding that it is the responsibility of the owner, or of 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 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 date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are 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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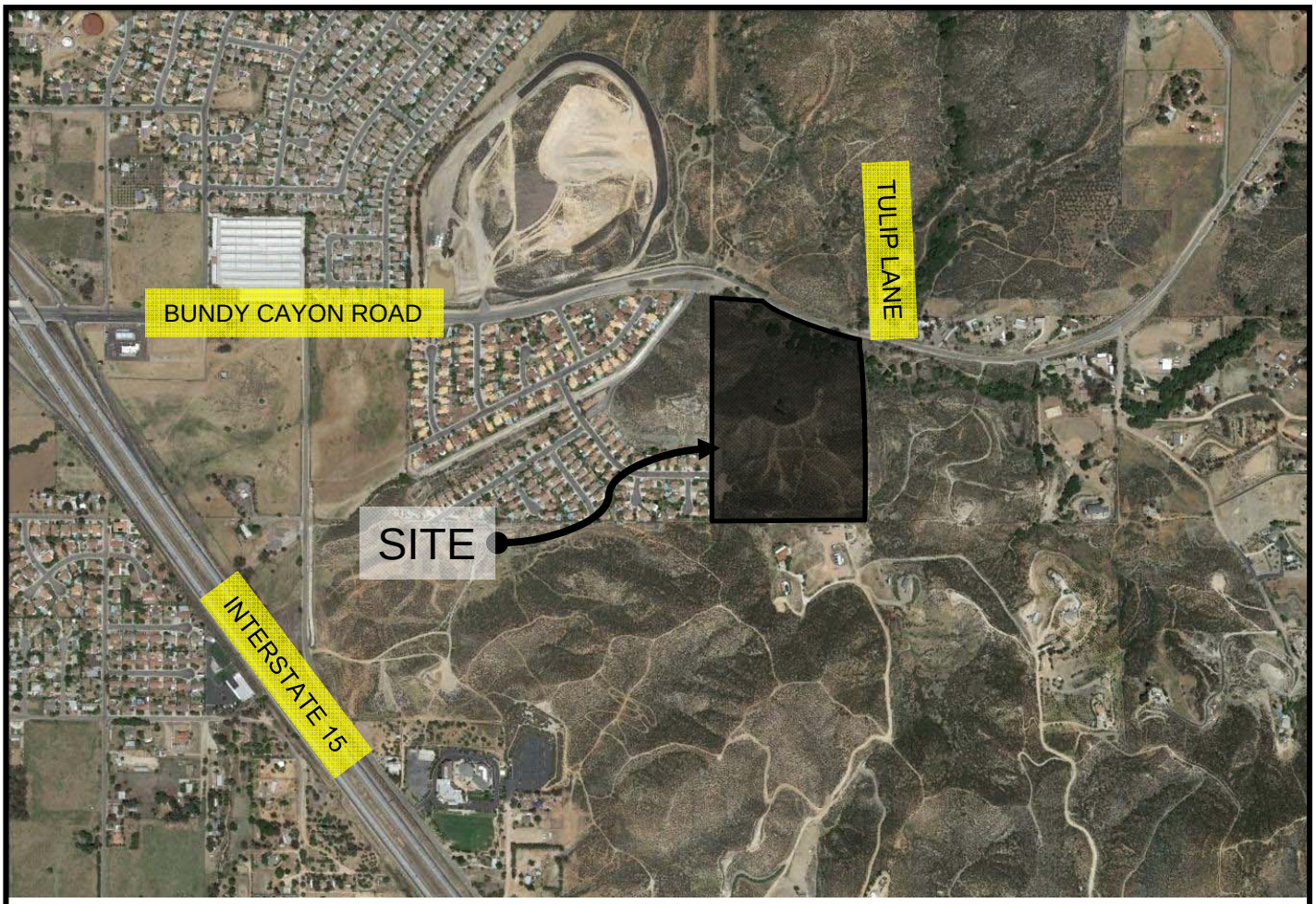


N

NO SCALE

SOURCE: Google Earth, 2016.

VICINITY MAP

GEOCON
WEST, INC.
GEOTECHNICAL CONSULTANTS
41571 CORNING PLACE SUITE 101 MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

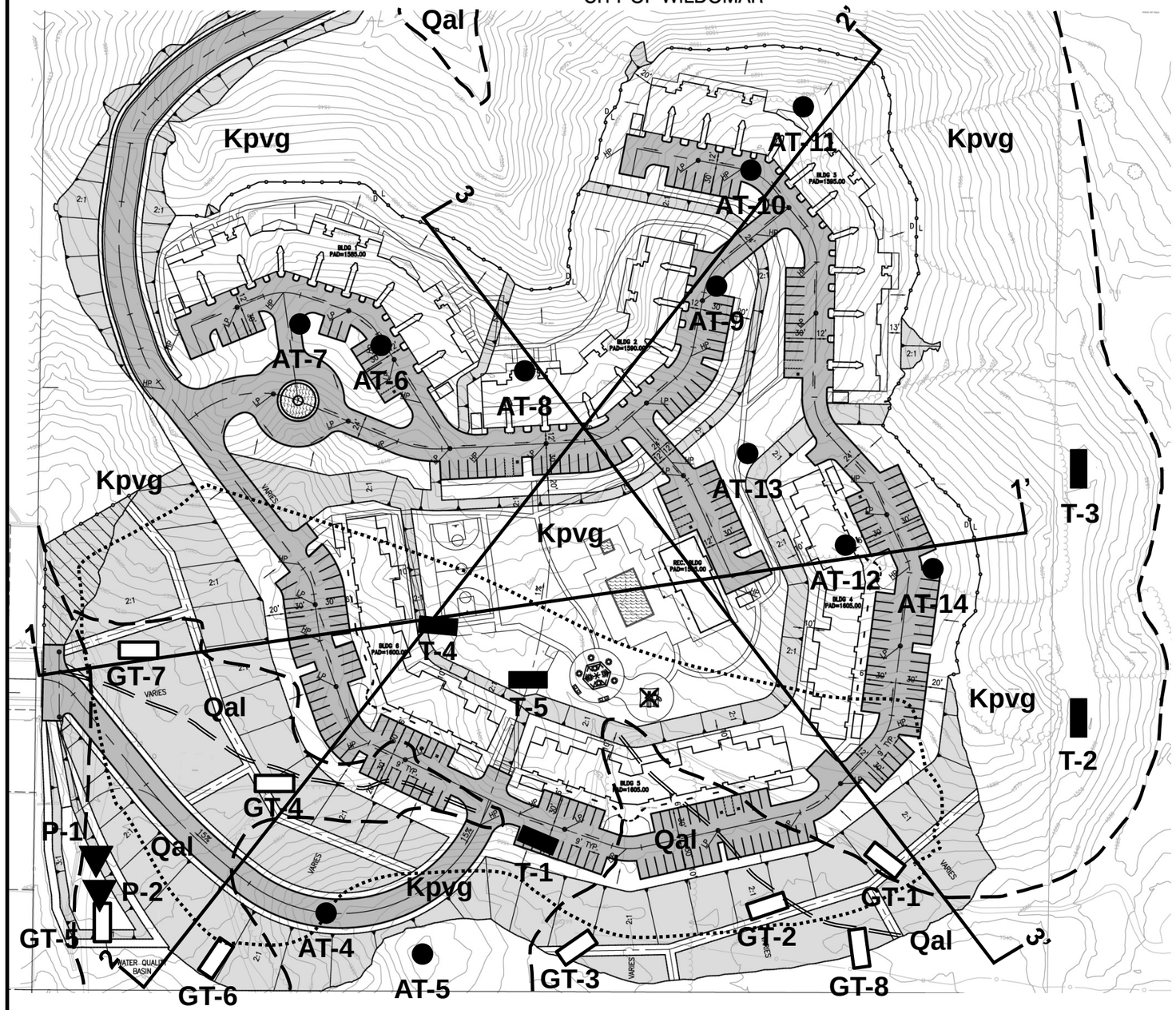
UPDATE GEOTECHNICAL INVESTIGATION
MUTI-FAMILY DEVELOPMENT
BUNDY CANYON ROAD NEAR TULIP LANE
WILDOMAR, CALIFORNIA

PDT		

FEBRUARY, 2016	PROJECT NO. T2145-22-02	FIG. 1

PRELIMINARY GRADING PLAN

CITY OF WILDOMAR



GEOCON LEGEND

Qal	ALLUVIUM
Kpvg	GRANITIC BEDROCK (GRANODIORITE)
	GEOLOGIC CONTACT
GT-8	TEST PIT (GEOCON, THIS REPORT)
P-2	PERCOLATION TEST(GEOCON, THIS REPORT)
3 3'	LOCATION OF CROSS-SECTION USED IN SLOPE STABILITY CALCULATIONS (GEOCON, THIS REPORT)
	APPROXIMATE ROCK DISPOSAL AREA
	PROPOSED CANYON SUBDRAIN
T-5	TEST PIT (GEOCON, 2003)
AT-14	TEST PIT (GEOCON, 2003)

SCALE: 1"=50'

0 50 100 150

SOURCE: Grant Becklund Civil Engineering

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PDT/ PDT

GEOTECHNICAL MAP

UPDATE GEOTECHNICAL INVESTIGATION
MULTI-FAMILY DEVELOPMENT
BUNDY CANYON ROAD NEAR YULIP LANE
WILDOMAR, CALIFORNIA

FEBRUARY, 2016	PROJECT NO. T2145-22-02	FIG. 2

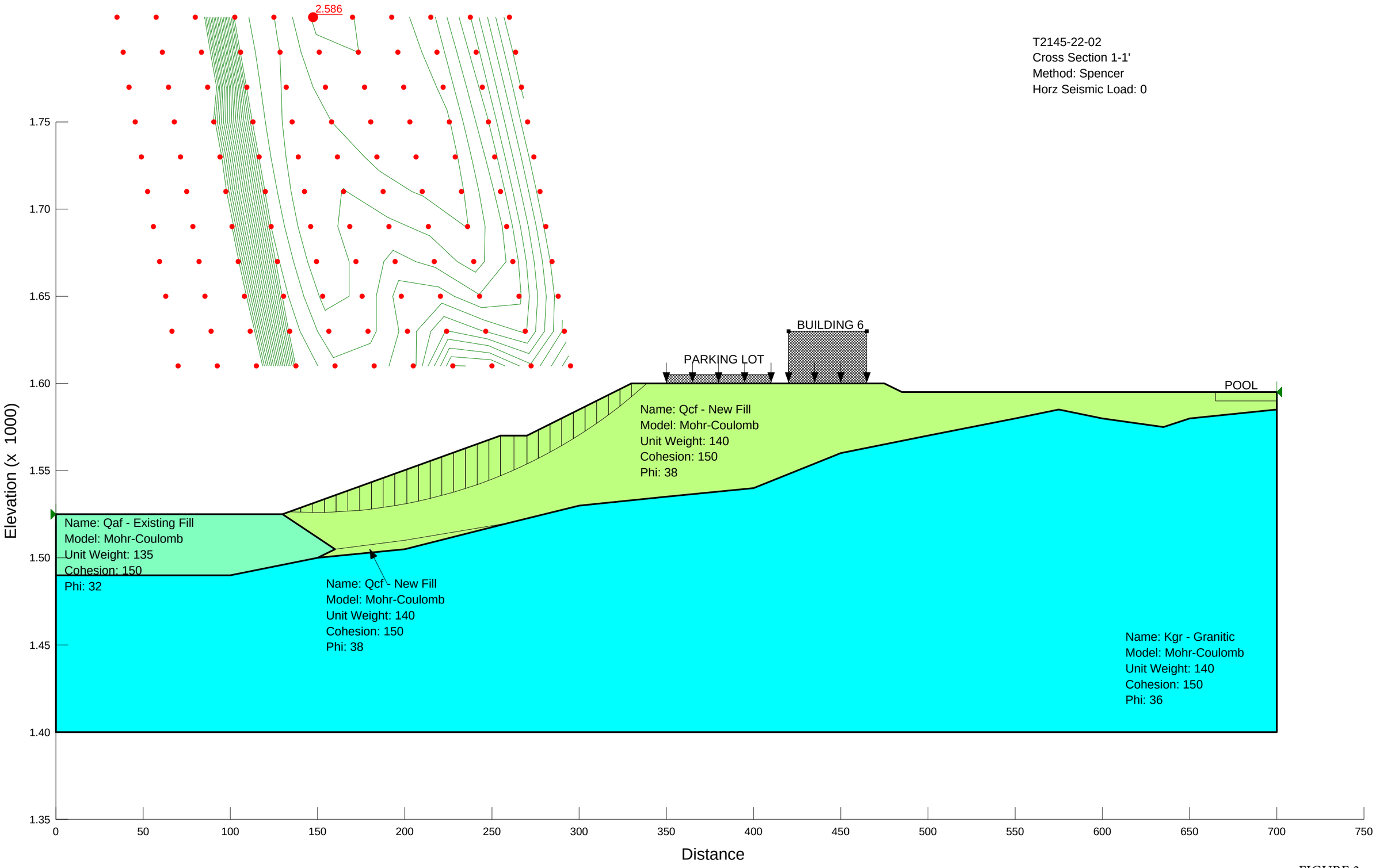

2.586
T2145-22-02
Cross Section 1-1'
Method: Spencer
Horz Seismic Load: 0
BUILDING 6
PARKING LOT
POOL
Name: Qcf - New Fill
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 38
Name: Qaf - Existing Fill
Model: Mohr-Coulomb
Unit Weight: 135
Cohesion: 150
Phi: 32
Name: Qcf - New Fill
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 38
Name: Kgr - Granitic
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 36
Elevation (x 1000)
1.75
1.70
1.65
1.60
1.55
1.50
1.45
1.40
1.35
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
Distance

FIGURE 3

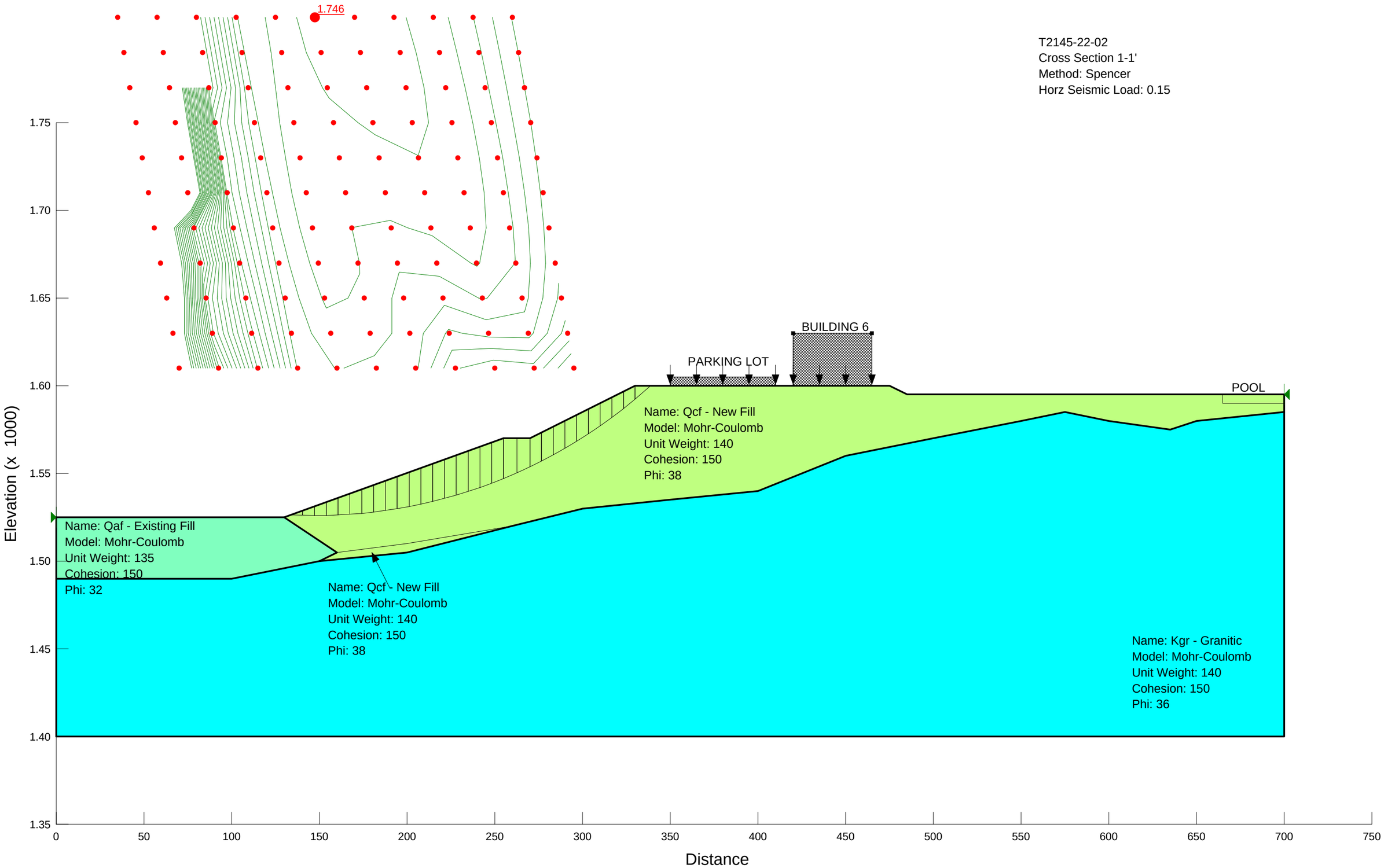
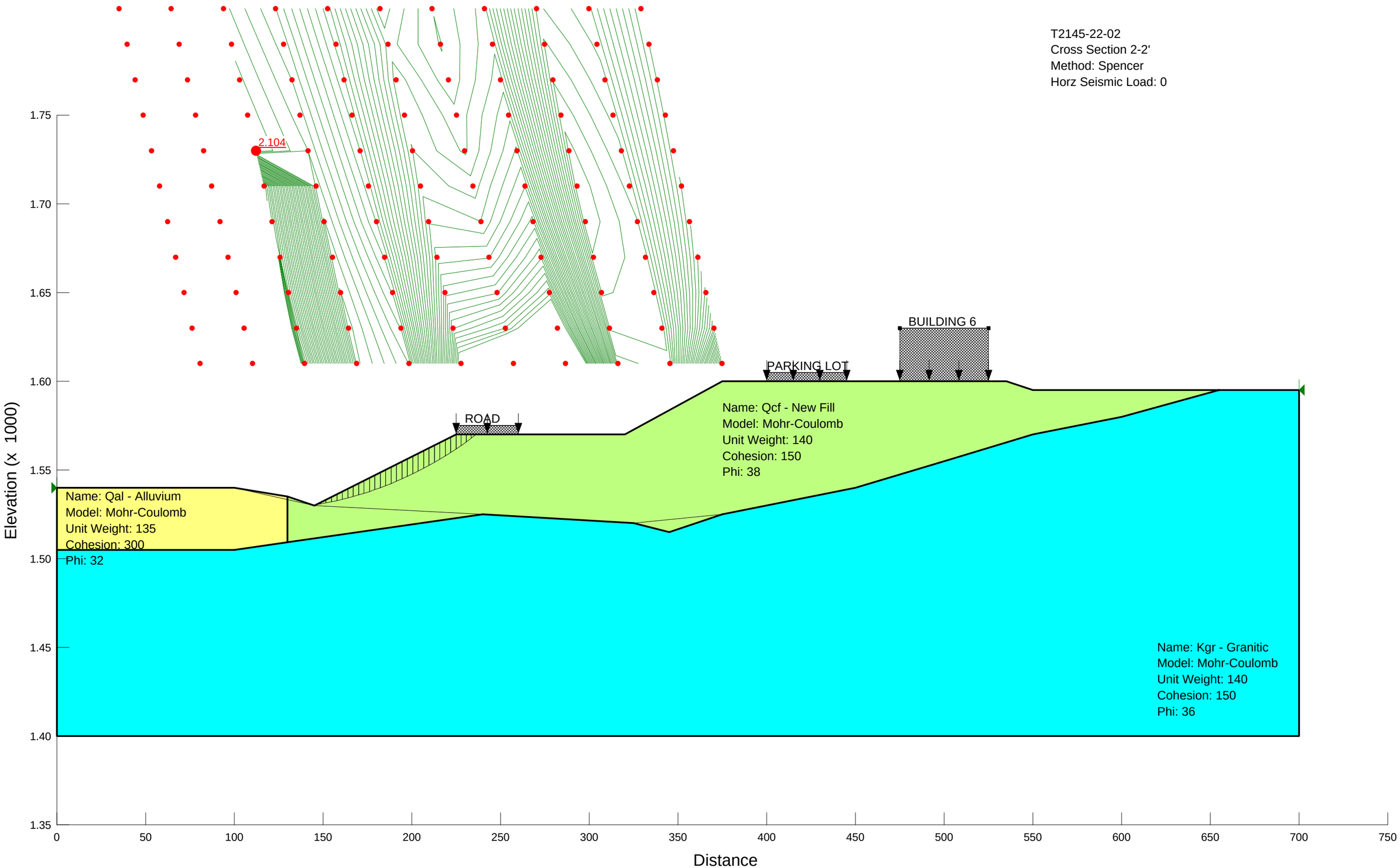

1.746
T2145-22-02
Cross Section 1-1'
Method: Spencer
Horz Seismic Load: 0.15
BUILDING 6
PARKING LOT
POOL
Name: Qcf - New Fill
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 38
Name: Qaf - Existing Fill
Model: Mohr-Coulomb
Unit Weight: 135
Cohesion: 150
Phi: 32
Name: Qcf - New Fill
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 38
Name: Kgr - Granitic
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 36
Elevation (x 1000)
1.75
1.70
1.65
1.60
1.55
1.50
1.45
1.40
1.35
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
Distance

FIGURE 4


T2145-22-02
Cross Section 2-2'
Method: Spencer
Horz Seismic Load: 0
2.104
BUILDING 6
PARKING LOT
ROAD
Name: Qcf - New Fill
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 38
Name: Qal - Alluvium
Model: Mohr-Coulomb
Unit Weight: 135
Cohesion: 300
Phi: 32
Name: Kgr - Granitic
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 36
Elevation (x 1000)
1.75
1.70
1.65
1.60
1.55
1.50
1.45
1.40
1.35
0
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100
150
200
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300
350
400
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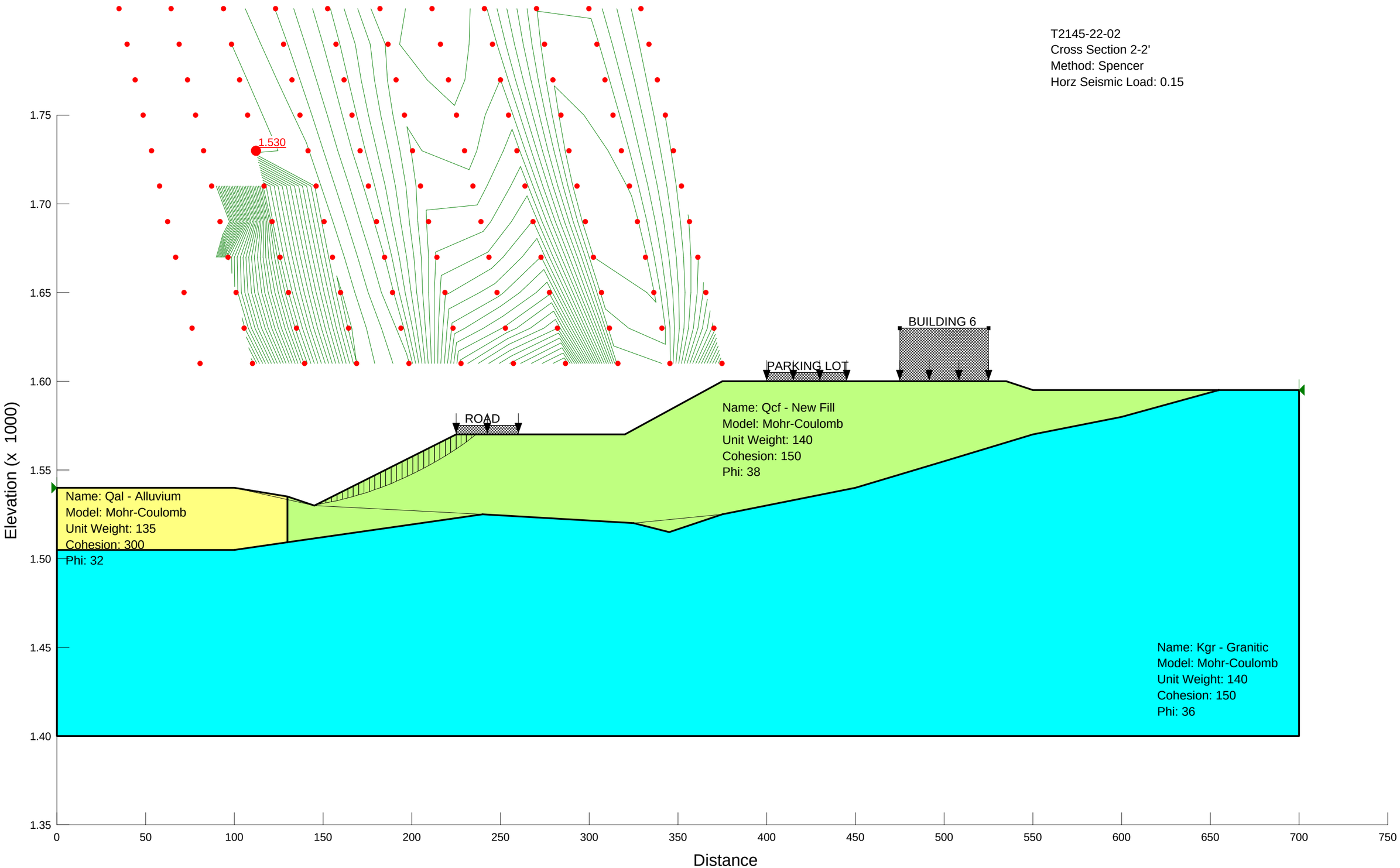
550
600
650
700
750
Distance

FIGURE 5


T2145-22-02
Cross Section 2-2'
Method: Spencer
Horz Seismic Load: 0.15
1.530
BUILDING 6
PARKING LOT
ROAD
Name: Qcf - New Fill
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 38
Name: Qal - Alluvium
Model: Mohr-Coulomb
Unit Weight: 135
Cohesion: 300
Phi: 32
Name: Kgr - Granitic
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 36
Elevation (x 1000)
1.75
1.70
1.65
1.60
1.55
1.50
1.45
1.40
1.35
0
50
100
150
200
250
300
350
400
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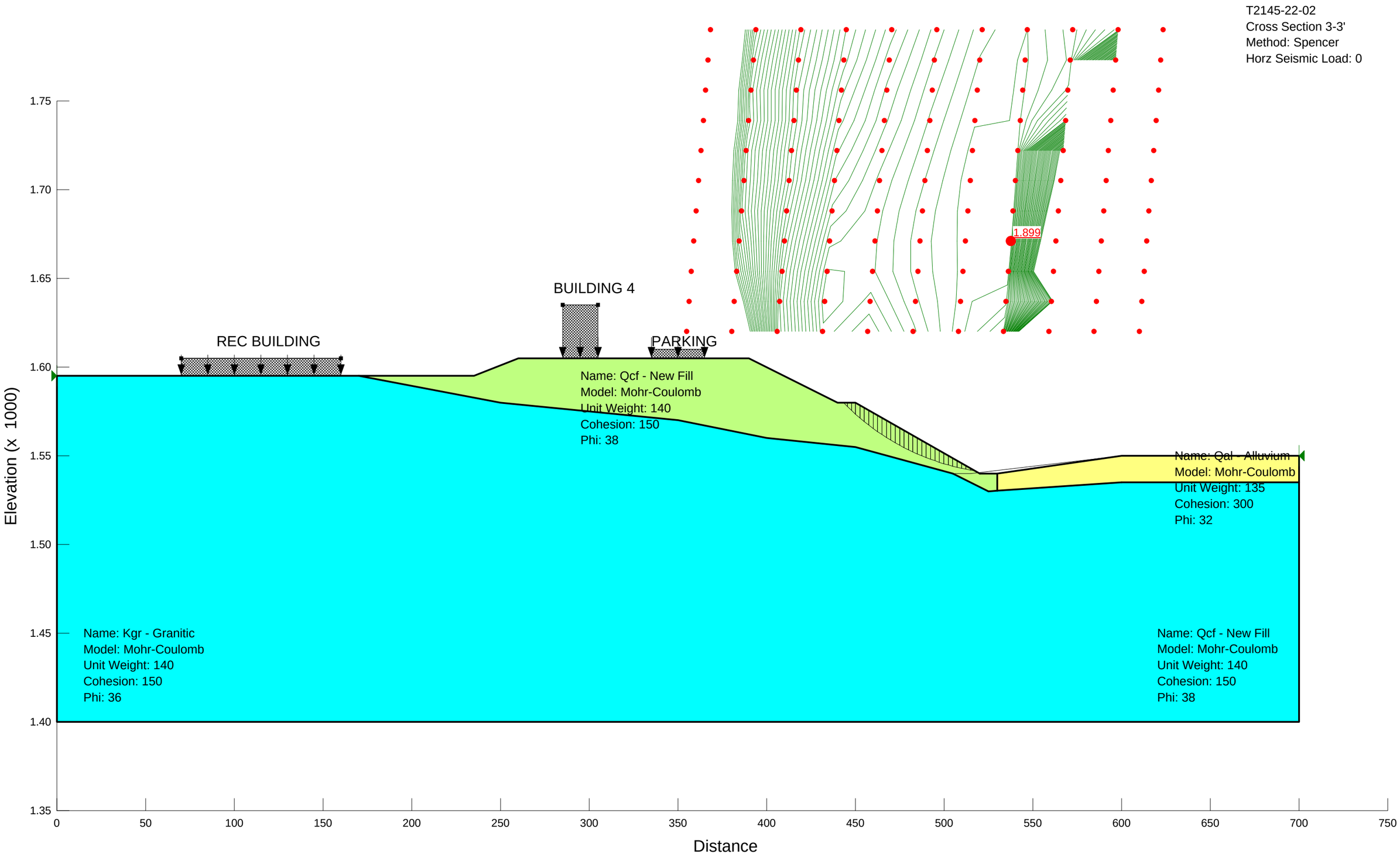
550
600
650
700
750
Distance

FIGURE 6


T2145-22-02
Cross Section 3-3'
Method: Spencer
Horz Seismic Load: 0
1.899
BUILDING 4
REC BUILDING
PARKING
Name: Qcf - New Fill
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 38
Name: Qal - Alluvium
Model: Mohr-Coulomb
Unit Weight: 135
Cohesion: 300
Phi: 32
Name: Kgr - Granitic
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 36
Name: Qcf - New Fill
Model: Mohr-Coulomb
Unit Weight: 140
Cohesion: 150
Phi: 38
Elevation (x 1000)
1.75
1.70
1.65
1.60
1.55
1.50
1.45
1.40
1.35
0
50
100
150
200
250
300
350
400
450
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650
700
750
Distance

FIGURE 7

T2145-22-02
Cross Section 3-3'
Method: Spencer
Horz Seismic Load: 0.15

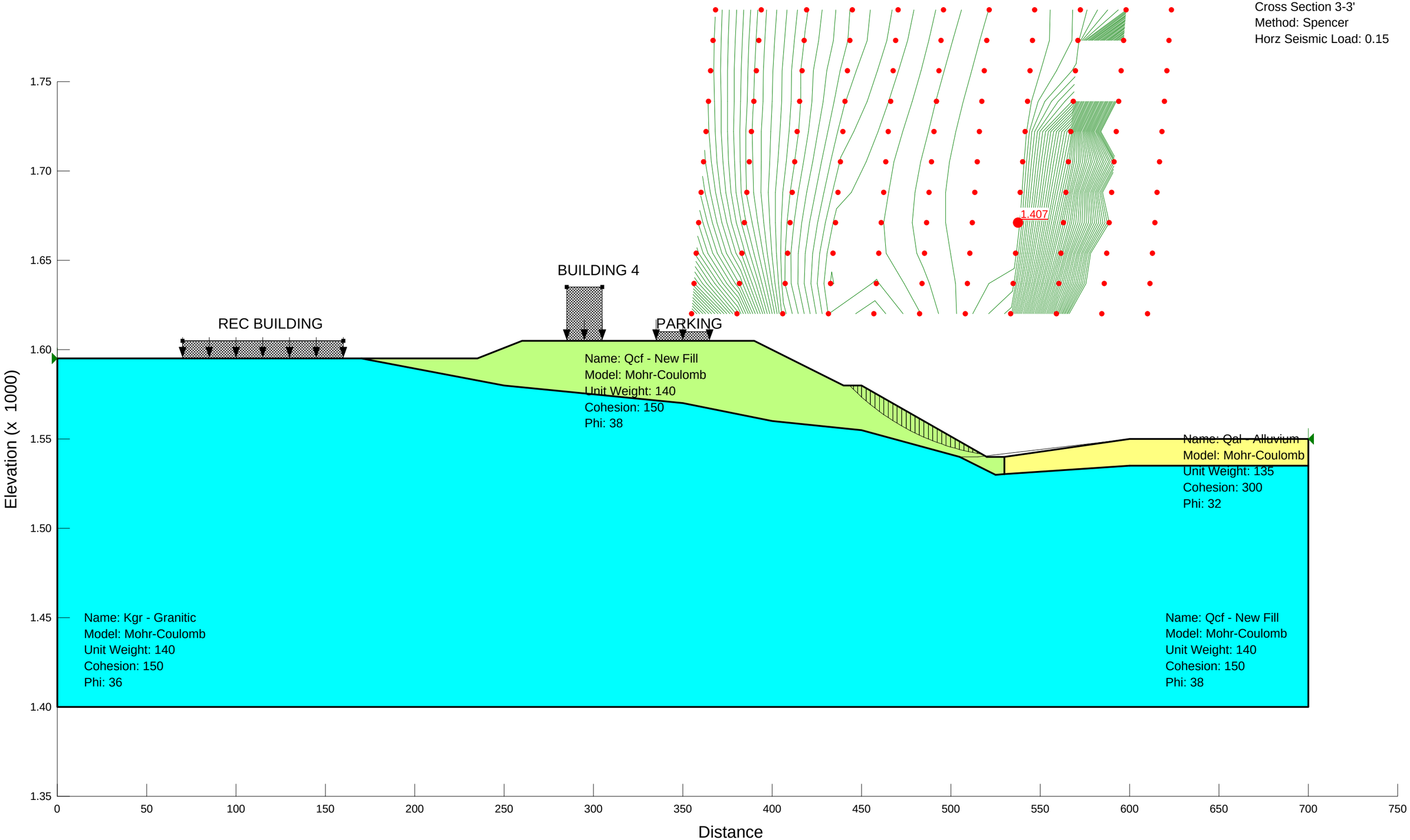
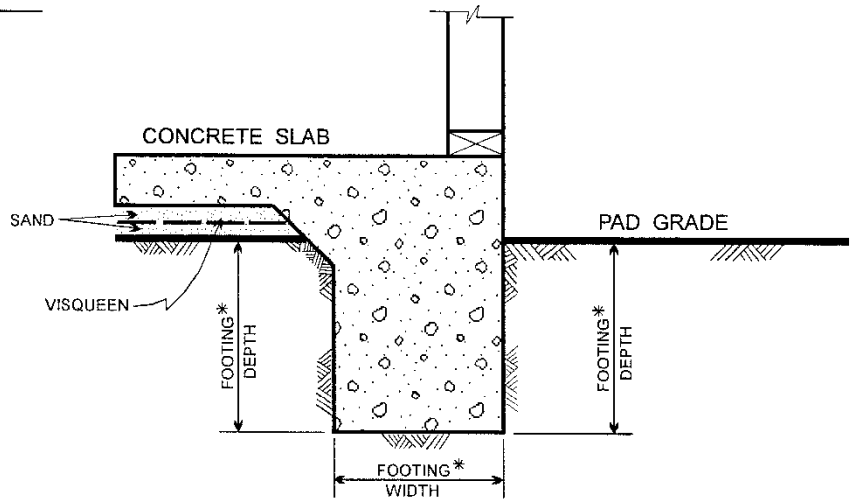
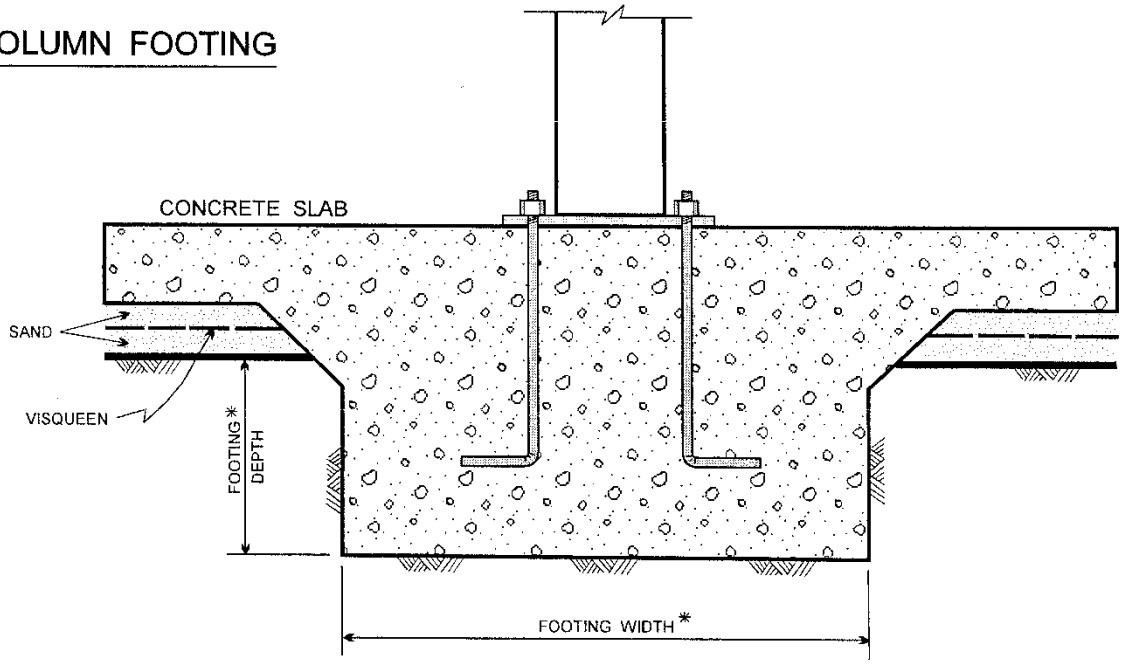


FIGURE 8

WALL FOOTING



COLUMN FOOTING



*……SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

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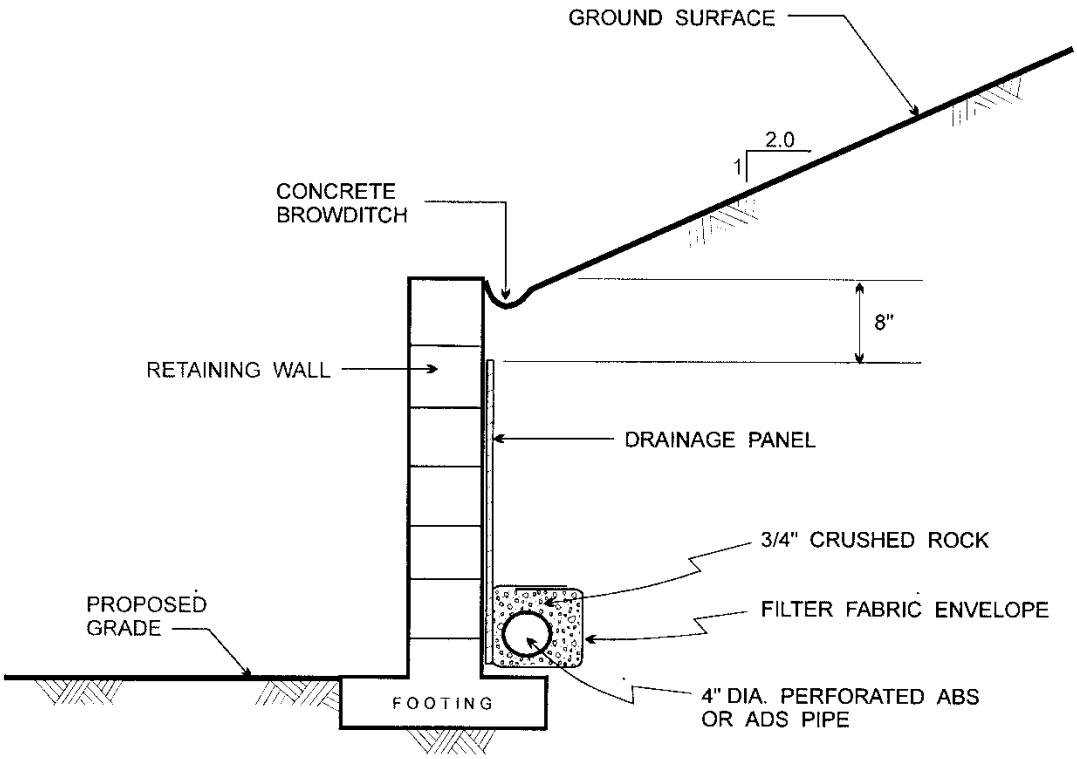
CER		

WALL / COLUMN FOOTING DETAIL

UPDAT EGEOTECHNICAL REPORT
MULTI-FAMILTY DEVELOPMENT
BYNDY CANYON ROAD NEAR TULIP LANE
WILDOMAR, CALIFORNIA

FEBRUARY, 2016	PROJECT NO. T2145-22-02	FIG. 9



NOTES:

1.....WALL DRAINAGE PANELS SHOULD CONSISTS OF MIRADRAIN 6000 OR EQUIVALENT

2.....FILTER FABRIC SHOULD CONSIST OF MIRAFI 140N OR APPROVED EQUIVALENT

3.....VOLUME OF CRUSHED ROCK SHOULD BE AT LEAST 1 CUBIC FOOT PER FOOT OF PIPE

4.....CONCRETE BROWDITCH RECOMMENDED FOR SLOPE HEIGHTS GREATER THAN 6 FEET

NO SCALE

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PDT		

WALL DRAINAGE DETAL

UPDATE GEOTECHNICAL INVESTIGATION
MULTI-FAMILY DEVELOPMENT
BUNDY CANYON ROAD NEAR TULIP LANE
WILDOMAR, CALIFORNIA

FEBRUARY, 2016	PROJECT NO. T2145-22-02	FIG. 10

PERCOLATION TEST REPORT							
Project Name:		Bundy Canyon			**Project No.:**		T2145-22-02
Test Hole No.:		P-1			**Date Excavated:**		2/4/2016
Length of Test Pipe:			15.6	inches	**Soil Classification:**		SM
Height of Pipe above Ground:			3.5	inches	**Presoak Date:**		2/4/2016
Depth of Test Hole:			12.1	inches	**Perc Test Date:**		2/5/2016
Check for Sandy Soil Criteria Tested by:				AMO	**Percolation Tested by:**		PDT
Water level measured from bottom of hole							
Sandy Soil Criteria Test							
Trial No.	**Time**	**Time Interval (min)**	**Total Elapsed Time (min)**	**Initial Water Level (in)**	**Final Water Level (in)**	**Δ in Water Level (inches)**	**Percolation Rate (min/inch)**
1	9:00 AM 9:25 AM	25	25	9.7	9.0	0.7	35.7
2	9:25 AM 9:50 AM	25	50	10.9	9.7	1.2	20.8
			Soil Criteria: Normal				
			Percolation Test				
Reading No.	**Time**	**Time Interval (min)**	**Total Elapsed Time (min)**	**Initial Water Head (in)**	**Final Water Head (in)**	**Δ in Water Level (inches)**	**Percolation Rate (min/inch)**
1	9:50 AM 10:20 AM	30	30	11.0	10.3	0.7	42.9
2	10:20 AM 10:50 AM	30	60	10.3	9.5	0.8	37.5
3	10:50 AM 11:20 AM	30	90	11.4	10.6	0.8	37.5
4	11:20 AM 11:50 AM	30	120	11.8	11.6	0.2	150.0
5	11:50 AM 12:20 PM	30	150	11.6	10.6	1.0	30.0
6	12:20 PM 12:50 PM	30	180	12.0	11.3	0.7	42.9
7	12:50 PM 1:20 PM	30	210	11.3	10.5	0.8	37.5
8	1:20 PM 1:50 PM	30	240	10.5	10.2	0.3	100.0
9	1:50 PM 2:20 PM	30	270	11.3	10.5	0.8	37.5
10	2:20 PM 2:50 PM	30	300	10.5	10.1	0.4	75.0
11	2:50 PM 3:20 PM	30	330	10.1	9.5	0.6	50.0
12	3:20 PM 3:50 PM	30	360	10.4	9.9	0.5	60.0
Infiltration Rate (in/hr):			0.2				
Radius of test hole (in):			6				**Figure A-11**
Average Head (in):			10.2				

PERCOLATION TEST REPORT							
Project Name:		Bundy Cayon			**Project No.:**		T2145-22-02
Test Hole No.:		P-2			**Date Excavated:**		2/4/2016
Length of Test Pipe:			16.5	inches	**Soil Classification:**		SM
Height of Pipe above Ground:			4	inches	**Presoak Date:**		2/4/2016
Depth of Test Hole:			12.5	inches	**Perc Test Date:**		2/5/2016
Check for Sandy Soil Criteria Tested by:				PDT	**Percolation Tested by:**		PDT
Water level measured from bottom of hole							
Sandy Soil Criteria Test							
Trial No.	**Time**	**Time Interval (min)**	**Total Elapsed Time (min)**	**Initial Water Level (in)**	**Final Water Level (in)**	**Δ in Water Level (inches)**	**Percolation Rate (min/inch)**
1	9:02 AM 9:27 AM	25	25	9.1	8.5	0.6	41.7
2	9:27 AM 9:52 AM	25	50	10.0	9.3	0.7	35.7
			Soil Criteria: Normal				
			Percolation Test				
Reading No.	**Time**	**Time Interval (min)**	**Total Elapsed Time (min)**	**Initial Water Level (in)**	**Final Water Level (in)**	**Δ in Water Level (inches)**	**Percolation Rate (min/inch)**
1	9:22 AM 10:22 AM	30	30	10.2	9.5	0.7	42.9
2	10:22 AM 10:52 AM	30	60	10.5	9.6	0.9	33.3
3	10:52 AM 11:22 AM	30	90	10.7	9.3	1.4	21.4
4	11:22 AM 11:52 AM	30	120	10.5	9.8	0.7	42.9
5	11:52 AM 12:22 PM	30	150	10.5	9.7	0.8	37.5
6	12:22 PM 12:52 PM	30	180	10.6	9.6	1.0	30.0
7	12:52 PM 1:22 PM	30	210	10.3	9.8	0.5	60.0
8	1:22 PM 1:52 PM	30	240	9.8	9.1	0.7	42.9
9	1:52 PM 2:22 PM	30	270	10.4	9.8	0.6	50.0
10	2:22 PM 2:52 PM	30	300	10.4	9.6	0.8	37.5
11	2:52 PM 3:22 PM	30	330	10.6	9.9	0.7	42.9
12	3:22 PM 3:52 PM	30	360	10.4	9.7	0.7	42.9
Infiltration Rate (in/hr):			0.3				
Radius of test hole (in):			6				**Figure A-12**
Average Head (in):			10.1				

APPENDIX A

APPENDIX A

EXPLORATORY EXCAVATIONS

We performed the field investigation on January 15, and February 4 and 5, 2016. Our subsurface exploration consisted of excavating 8 geotechnical test pits along the southern portion of the site, and hand excavating two percolation pits within the proposed basin in the southwest corner of the site. Excavation and presaturation of the percolation test holes was performed on February 4, 2016, and percolation testing was performed on February 5, 2016 in accordance with *Riverside County Flood Control and Water Conservation District Low Impact Development Handbook Appendix A (Handbook)*. The test pits were excavated with a backhoe to depths of up to 15 feet. Bulk samples of disturbed soils were transported to our laboratory for testing.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A-1 through A-8. Percolation test data is presented on Figures A-9 and A-10. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. Previous site exploration logs are in *Appendix C*. The approximate locations of the test pits and percolations pits, as well as previous site explorations are shown the *Geotechnical Map*, Figure 2.

PROJECT NO. T2145-22-02

TEST PIT GT-1

ELEV. (MSL.) **1552** DATE COMPLETED **01/15/2016**

EQUIPMENT **BACK HOE** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Silty SAND, loose, slightly moist, brown; fine to coarse sand; some gravel; sparse shubs at surface			
2					**GRANITIC BEDROCK (Kpvg)** Completely weathered, reddish brown and yellowish brown, weak, GRANODIORITE; excavates as a sand with trace gravel			
4	GT-1@4.5				-Becomes highly weathered; moderately weak			
6					-Becomes moderately weathered; slow advance			
8					-Becomes lightly weathered			

TOTAL DEPTH 8 FEET
Groundwater not encountered
No caving
Backfilled with cuttings 1/15/2016

Figure A-1,
Log of Test Pit GT-1, Page 1 of 1

T2145-22-02 APPENDIX A TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2145-22-02

TEST PIT GT-2

ELEV. (MSL.) **1549** DATE COMPLETED **01/15/2016**

EQUIPMENT **BACK HOE** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Silty SAND, loose, slightly moist, strong brown; fine to coarse sand; trace gravel; sparse shubs at surface			
2	GT-2@1'							
4				SM	**ALLUVIUM (Qal)** Silty SAND, loose, slightly moist, yellowish brown; fine to coarse sand; trace clay; moderate porosity (up to 1/8 inch diam.)			
6								
8								
10					**GRANITIC BEDROCK (Kpvg)** Completely weathered, yellowish brown, weak, GRANODIORITE; excavates as a gravelly sand; some clay alteration -Becomes moderately weathered			
12								

TOTAL DEPTH 12 FEET
Groundwater not encountered
No caving
Backfilled with cuttings 1/15/2016

Figure A-2,
Log of Test Pit GT-2, Page 1 of 1

T2145-22-02 APPENDIX A TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. T2145-22-02

TEST PIT GT-3

ELEV. (MSL.) **1565** DATE COMPLETED **01/15/2016**

EQUIPMENT **BACK HOE** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Silty SAND, loose, slightly moist, dark brown; fine sand; some medium and coarse sand; some gravel; sparse shubs at surface			
2								
4				SM	**ALLUVIUM (Qal)** Silty SAND, medium dense, slightly moist, yellowish brown; fine to coarse sand; moderate porosity (up to 1/2 inch diam.); root hairs			
6								
8					**GRANITIC BEDROCK (Kpvg)** Completely weathered, yellowish brown, weak, GRANODIORITE; excavates as a gravelly sand; -Becomes moderately weathered			
10					TOTAL DEPTH 10 FEET Groundwater not encountered No caving Backfilled with cuttings 1/15/2016			

Figure A-3,
Log of Test Pit GT-3, Page 1 of 1

T2145-22-02 APPENDIX A TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2145-22-02

TEST PIT GT-4

ELEV. (MSL.) **1510** DATE COMPLETED **01/15/2016**

EQUIPMENT **BACK HOE** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0								
				SM	**ALLUVIUM (Qal)** Silty SAND, loose, moist, brown; fine to coarse sand; roots; sparse shubs at surface			
2								
4					-Becomes light brown; some gravel; moderate porosity (up to 1/8 inch diam.)			
6								
8	GT-4@8				-Becomes brownish yellow; fine to medium sand; non porous			8.8
10								
12					**GRANITIC BEDROCK (Kpvg)** Completely weathered, olive/light gray, weak, GRANODIORITE; excavates as a gravelly sand with silt -Excavates as a sandy gravel with cobbles			
14					TOTAL DEPTH 14 FEET Groundwater not encountered No caving Backfilled with cuttings 1/15/2016			

Figure A-4,
Log of Test Pit GT-4, Page 1 of 1

T2145-22-02 APPENDIX A TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2145-22-02

TEST PIT GT-5

ELEV. (MSL.) **1518** DATE COMPLETED **01/15/2016**

EQUIPMENT **BACK HOE** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ALLUVIUM (Qal)** Silty SAND, loose, slightly moist, brown; fine to coarse sand; some gravel; roots; sparse shubs at surface			
2								
4					-Becomes medium dense			
6								
8					**GRANITIC BEDROCK (Kpvg)** Completely weathered, yellowish brown, weak, GRANODIORITE; excavates as a sandy gravel			
10					-Becomes highly weathered			
12								
14					-Becomes moderately weathered; slow advance			
					TOTAL DEPTH 15 FEET Groundwater not encountered No caving Backfilled with cuttings 1/15/2016			

Figure A-5,
Log of Test Pit GT-5, Page 1 of 1

T2145-22-02 APPENDIX A TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL ... STANDARD PENETRATION TEST ... DRIVE SAMPLE (UNDISTURBED)

... DISTURBED OR BAG SAMPLE ... CHUNK SAMPLE ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. T2145-22-02

TEST PIT GT-6

ELEV. (MSL.) **1528** DATE COMPLETED **01/15/2016**

EQUIPMENT **BACK HOE** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ALLUVIUM (Qal)** Silty SAND, loose, moist, dark brown; fine to coarse sand; sparse shubs at surface			
2					-Becomes slightly moist, brownish yellow; some gravel			
4								
6								
8					-Slight increase in gravel content			
10								
12								
14								
					TOTAL DEPTH 15 FEET Groundwater not encountered No caving Backfilled with cuttings 1/15/2016			

Figure A-6,
Log of Test Pit GT-6, Page 1 of 1

T2145-22-02 APPENDIX A TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. T2145-22-02

TEST PIT GT-7

ELEV. (MSL.) **1513** DATE COMPLETED **01/15/2016**

EQUIPMENT **BACK HOE** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ALLUVIUM (Qal)** Silty SAND, loose, slightly moist, brown; fine to coarse sand; some gravel; sparse shubs at surface			
2								
4					**GRANITIC BEDROCK (Kpvg)** Completely weathered, black and white, weak, GRANODIORITE; excavates as a silty fine to medium sand; slightly finer-grained than in other excavations -Becomes moderately weathered			
6								
8					TOTAL DEPTH 8 FEET Groundwater not encountered No caving Backfilled with cuttings 1/15/2016			

Figure A-7,
Log of Test Pit GT-7, Page 1 of 1

T2145-22-02 APPENDIX A TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. T2145-22-02

TEST PIT GT-8

ELEV. (MSL.) **1550** DATE COMPLETED **01/15/2016**

EQUIPMENT **BACK HOE** BY: **P. THERIAULT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Silty SAND, loose, moist, dark brown; fine to coarse sand; sparse shubs at surface			
2				SM	**ALLUVIUM (Qal)** Silty SAND, loose, slightly moist, reddish brown; fine to coarse sand; root hairs; moderate porosity (up to 1/8 inch diam.) -Becomes medium dense			
4								
6								
8					**GRANITIC BEDROCK (Kpvg)** Completely weathered, olive to pale yellow, weak, GRANODIORITE; excavates as a silty sand with clay -Becomes highly weathered; brownish yellow; excavates as a gravelly sand -Becomes moderatly weathered; finer grained			
10								
12					TOTAL DEPTH 12 FEET Groundwater not encountered No caving Backfilled with cuttings 1/15/2016			

Figure A-8,
Log of Test Pit GT-8, Page 1 of 1

T2145-22-02 APPENDIX A TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. T2145-22-02

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TEST PIT P-1** ELEV. (MSL.) **1515** DATE COMPLETED EQUIPMENT **BACK HOE** BY: **P. THERIAULT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**ALLUVIUM (Qal)** Silty SAND, loose, slightly moist, brown; fine to coarse sand; little gravel; sparse shubs at surface			
	P-1@1							

TOTAL DEPTH 1.5 FEET
Groundwater not encountered
No caving
Backfilled with cuttings 2/05/2016

Figure A-9,
Log of Test Pit P-1, Page 1 of 1

T2145-22-02 APPENDIX A TEST PIT LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. T2145-22-02

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TEST PIT P-2** ELEV. (MSL.) **1512** DATE COMPLETED EQUIPMENT **BACK HOE** BY: **P. THERIAULT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	P-2@1			SM	**ALLUVIUM (Qal)** Silty SAND, loose, slightly moist, brown; fine to coarse sand; little gravel; sparse shubs at surface			

TOTAL DEPTH 1 FOOT
Groundwater not encountered
No caving
Backfilled with cuttings 2/05/2016

Figure A-10,
Log of Test Pit P-2, Page 1 of 1

T2145-22-02 APPENDIX A TEST PIT LOGS.GPJ

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with current, generally accepted test methods of ASTM International (ASTM) or other suggested procedures. We analyzed selected soil samples maximum dry density and optimum moisture content, direct shear strength, corrosion, grain size analysis, and R-Value. The results of the laboratory tests are presented on Figures B1 through B3. Additional laboratory data from previous investigations are presented in *Appendix C*.

SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS ASTM D1557

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% of dry wt.)
GT-4 @ 8-9’	Silty SAND, brown	135.4	9.3

SUMMARY OF CORROSIVITY TEST RESULTS

Sample No.	Chloride Content (ppm)	Sulfate Content (%)	pH	Resistivity (ohm-centimeter)
GT-2 @ 1-2’	86	0.001	6.4	3,800

Chloride content determined by California Test 422.
Water-soluble sulfate determined by California Test 417.
Resistivity and pH determined by Caltrans Test 643.

SUMMARY OF LABORATORY R-VALUE TEST RESULTS ASTM D2844

Sample No.	R-Value
GT-2 @ 1-2’	24

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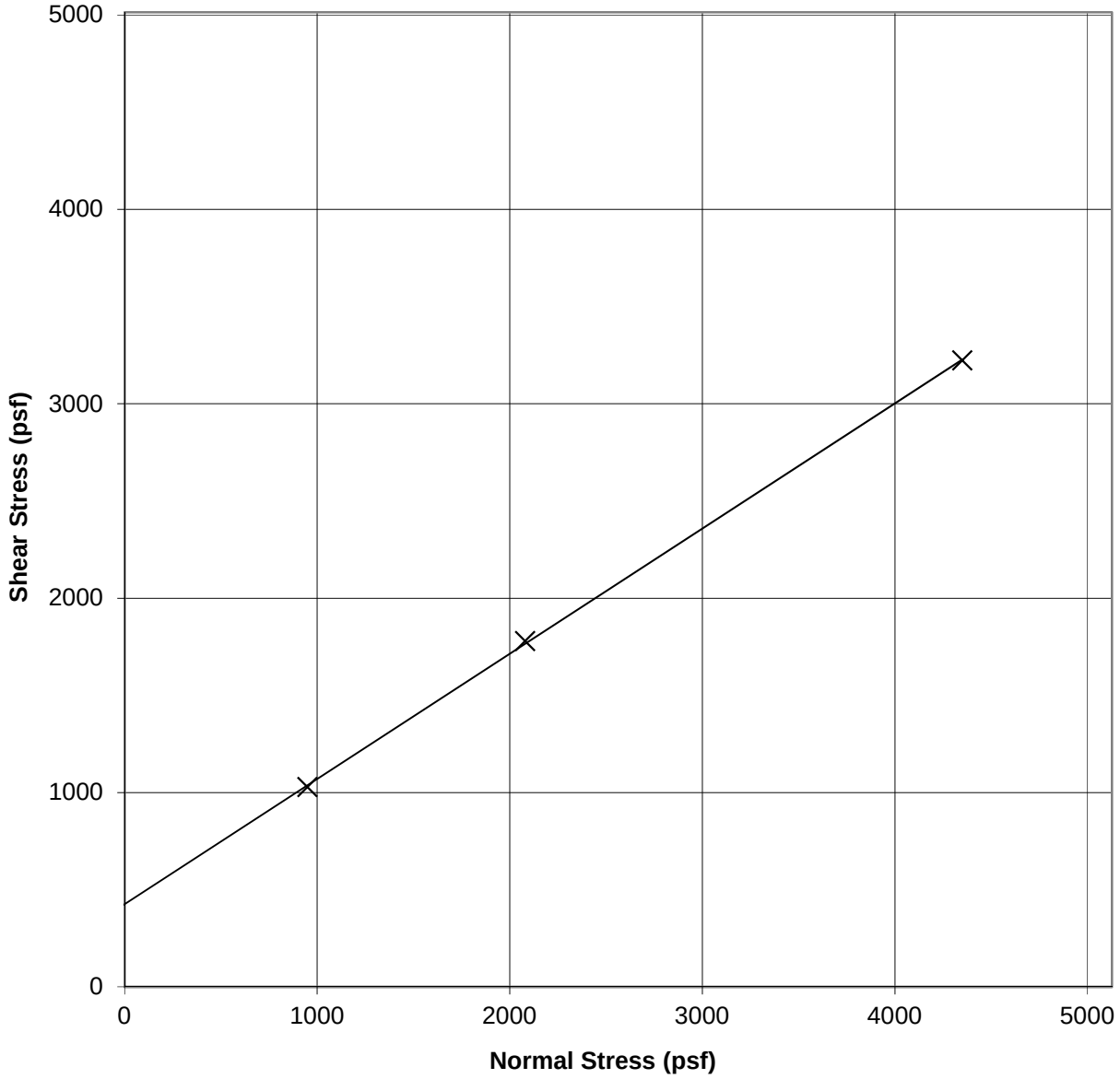
GEOTECHNICAL CONSULTANTS
41571 CORNING PLACE SUITE 101 MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

PDT

LABORATORY TEST RESULTS

UPDATE GEOTECHNICAL INVESTIGATION
MULTI-FAMILY DEVELOPMENT
BUNDY CANYON ROAD NEAR TULIP LANE
WILDOMAR, CALIFORNIA



SAMPLE ID	SOIL TYPE	INITIAL DRY DENSITY (pcf)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)	C (psf)	ϕ (deg)
*GT-4 @ 8'	SM	122.2	8.8	13.8	430	33

*Sample remolded to approximately 90% of the test maximum dry density at optimum moisture content.

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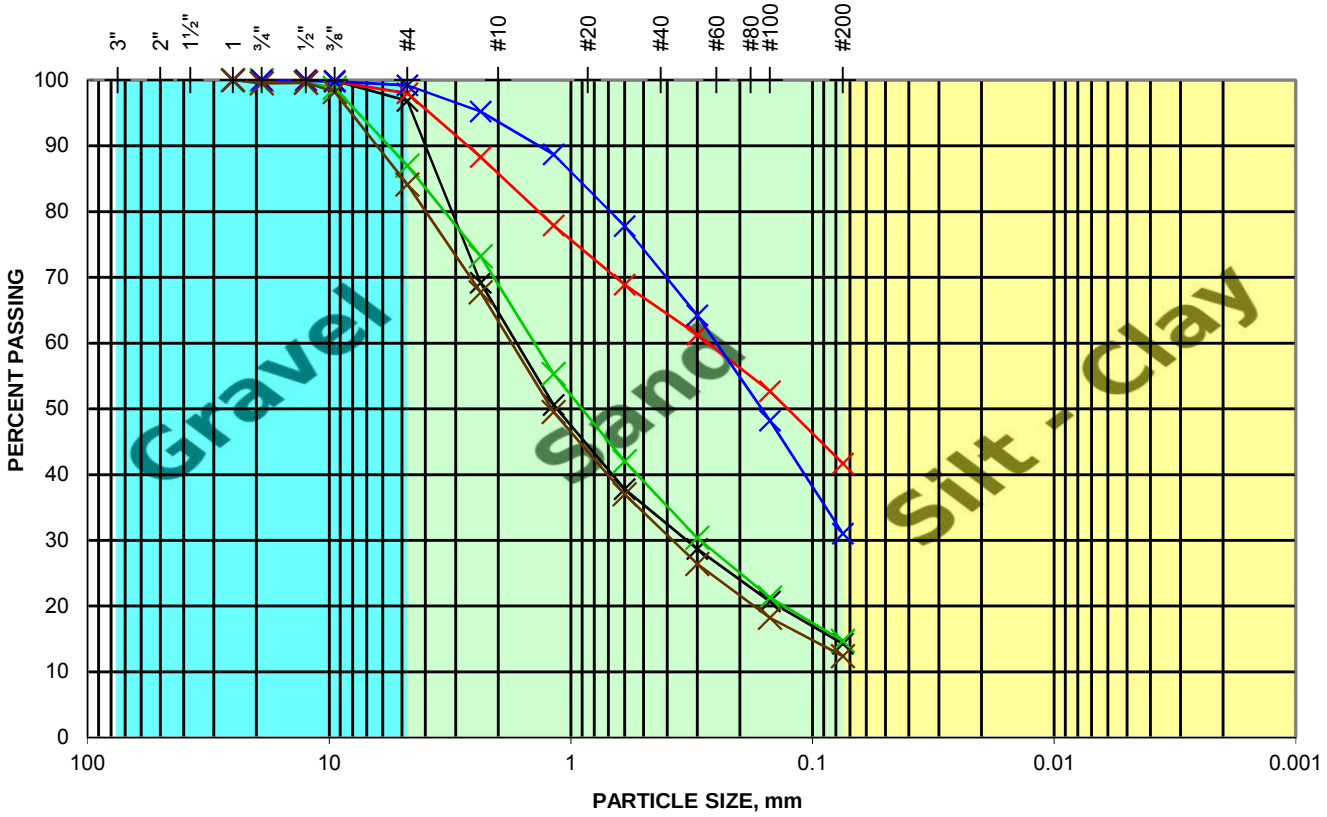
PDT

DIRECT SHEAR TEST RESULTS

UIPDATE GEOTECHNICAL INVESTIGATION
MULTI-FAMILY DEVELOPMENT
BUNDY CANYON ROAD NEAR TULIP LANE
WILDOMAR, CALIFORNIA

FEBRUARY, 2016 | PROJECT NO. T2145-22-02 | FIG B-2



SAMPLE ID	SAMPLE DESCRIPTION
GT-1 @ 4.5'	SM - Silty SAND, trace Gravel
GT-2 @ 1'	SM - Silty SAND, trace Gravel
GT-4 @ 8'	SM - Silty SAND, trace Gravel
P-1 @ 1'	SM - Silty SAND, little Gravel
P-2 @ 1'	SM - Silty SAND, little Gravel

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PHONE 951-304-2300 FAX 951-304-2392

PDT

GRAIN SIZE DISTRIBUTION

UPDATE GEOTECHNICAL INVESTIGATION
MUTI-FAMILY DEVELOPMENT
BUNDY CANYON ROAD NEAR TULIP LANE
WILDOMAR, CALIFORNIA

FEBRUARY, 2016	PROJECT NO. T2145-22-01	FIG. B-3

APPENDIX C

APPENDIX C

PREVIOUS EXPLORATION LOGS AND LABORATORY TEST DATA

FOR

MULI-FAMILY DEVELOPMENT
BUNDY CANYON ROAD NEAR TULIP LANE
WILDOMAR, CALIFORNIA

PROJECT NO. T2145-22-02

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed on July 1 and 2, 2003, and consisted of a site reconnaissance, the advancement of three exploratory excavations and 11 air-track borings. The exploratory excavations were made with a D8 dozer. The air-track borings were excavated to depths ranging from 40 to 80 feet below existing grade using an Ingersoll-Rand ECM-370 air-track drill rig equipped with a 4-inch bit. An additional 3 air-track borings and two exploratory excavations were excavated on the easterly adjacent tract (Tract No. 29426). During advancement of the air-track borings, the rate of penetration was timed. The rates of penetration as a function of depth (for all 14 borings) are plotted on Figures A-1 through A-14. Based on information provided by Edick & Watt, Inc., the drill penetration rates can be related to the rock rippability approximately as noted below:

Drill Rates	Rippability Characteristics
0-20 sec/ft	Rippable
20-30 sec/ft	Marginally Rippable
+30 sec/ft	Non-Rippable

Previous investigations by Byerly (2001 and 2002) on the easterly adjacent property included borings and trench excavation logs which are provided in Appendix C. The soil conditions encountered in the Geocon excavations were visually examined, classified and logged in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D2488). The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the exploratory excavations are shown on the Geologic Map, Figure 2.

TRACT NO. 26372

AIR TRACK BORING AT-4

Elevation - 1570Feet (MSL)

Date 07-02-2003 - Equipment: IR 370

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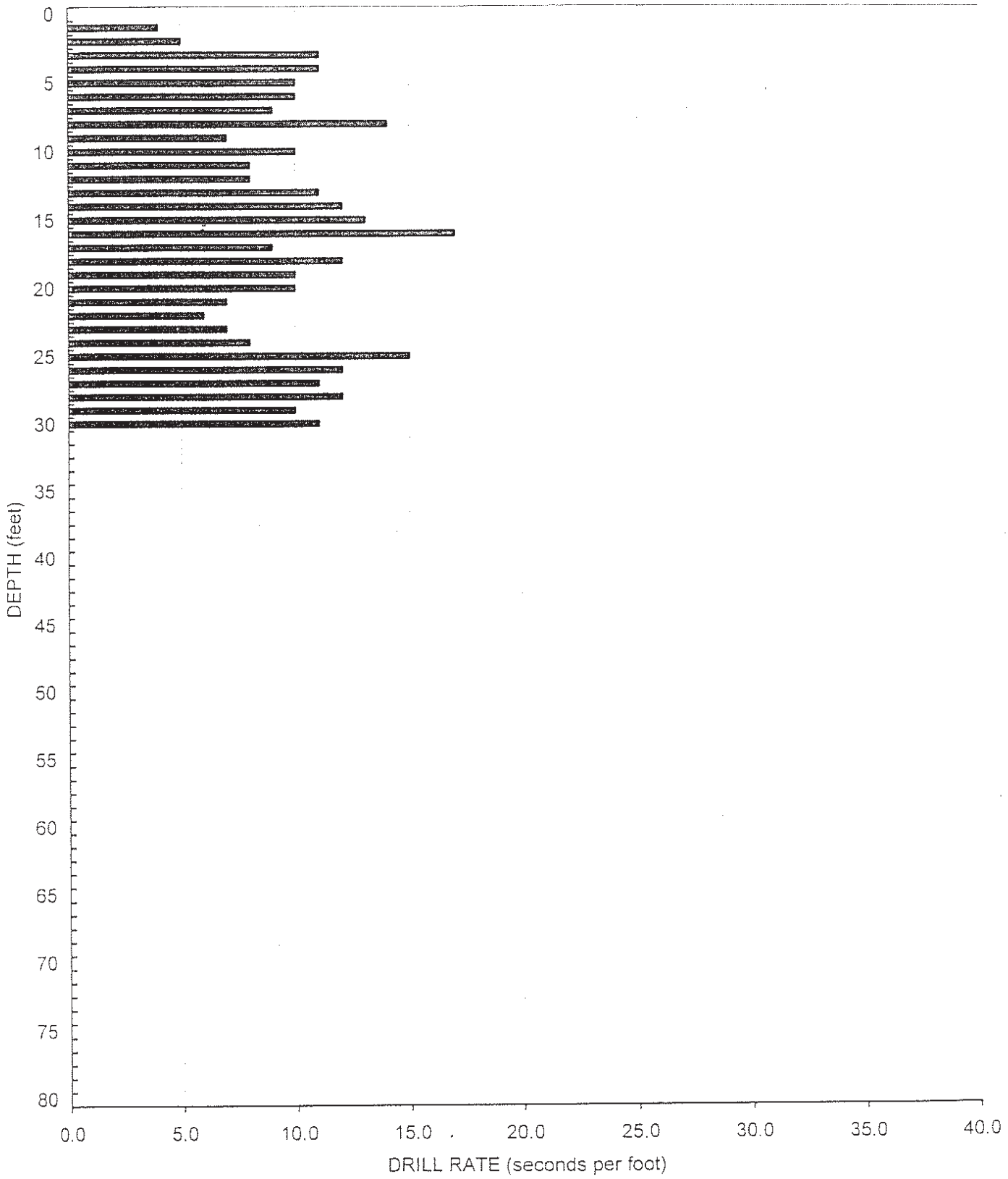

AT04.xls

FIGURE A-4

FIGURE C-1

TRACT NO. 26372

AIR TRACK BORING AT-5

Elevation - 1590 Feet (MSL)

Date 07-02-2003 - Equipment: IR 370

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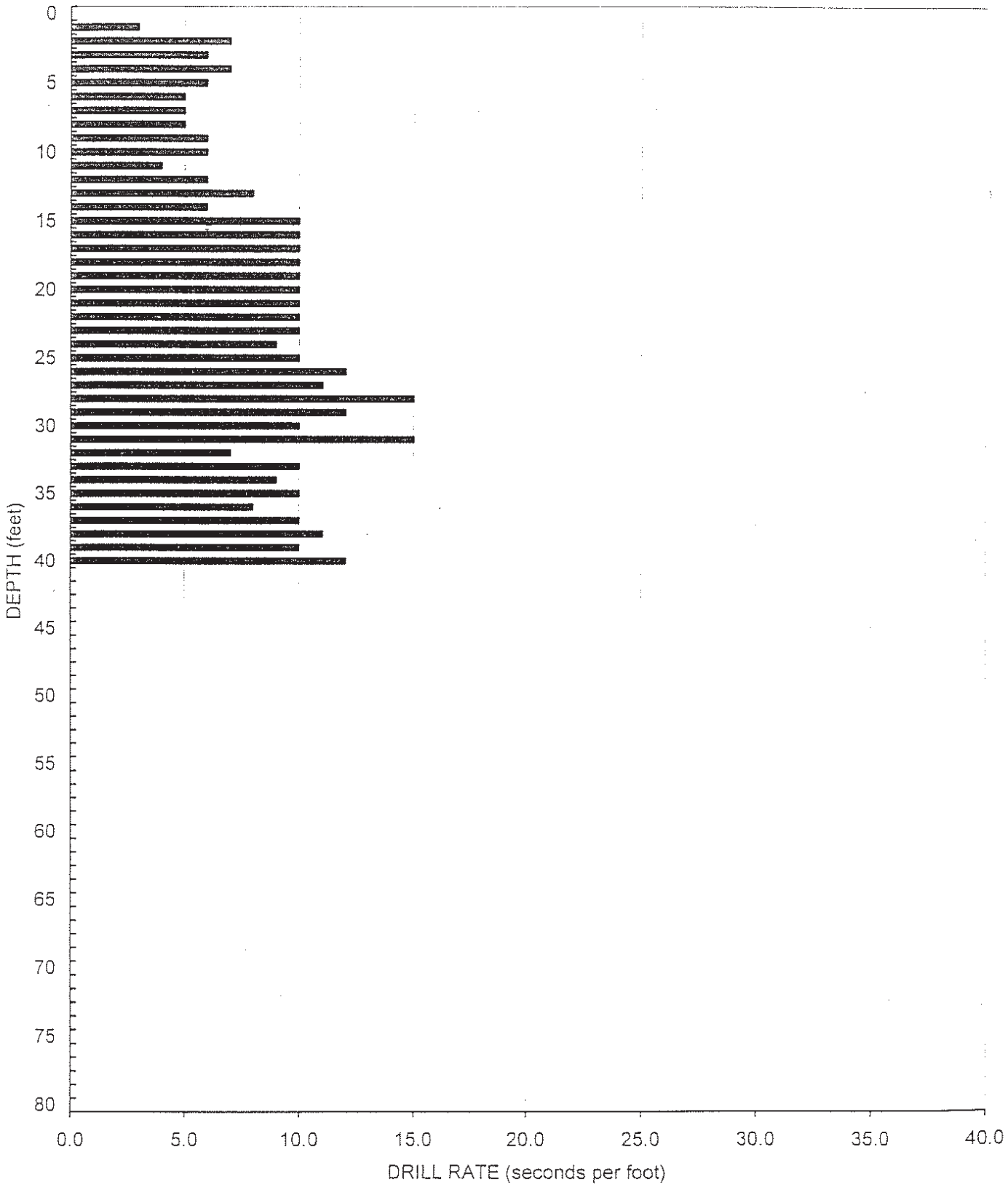


AT05.xls

FIGURE A-5

FIGURE C-2

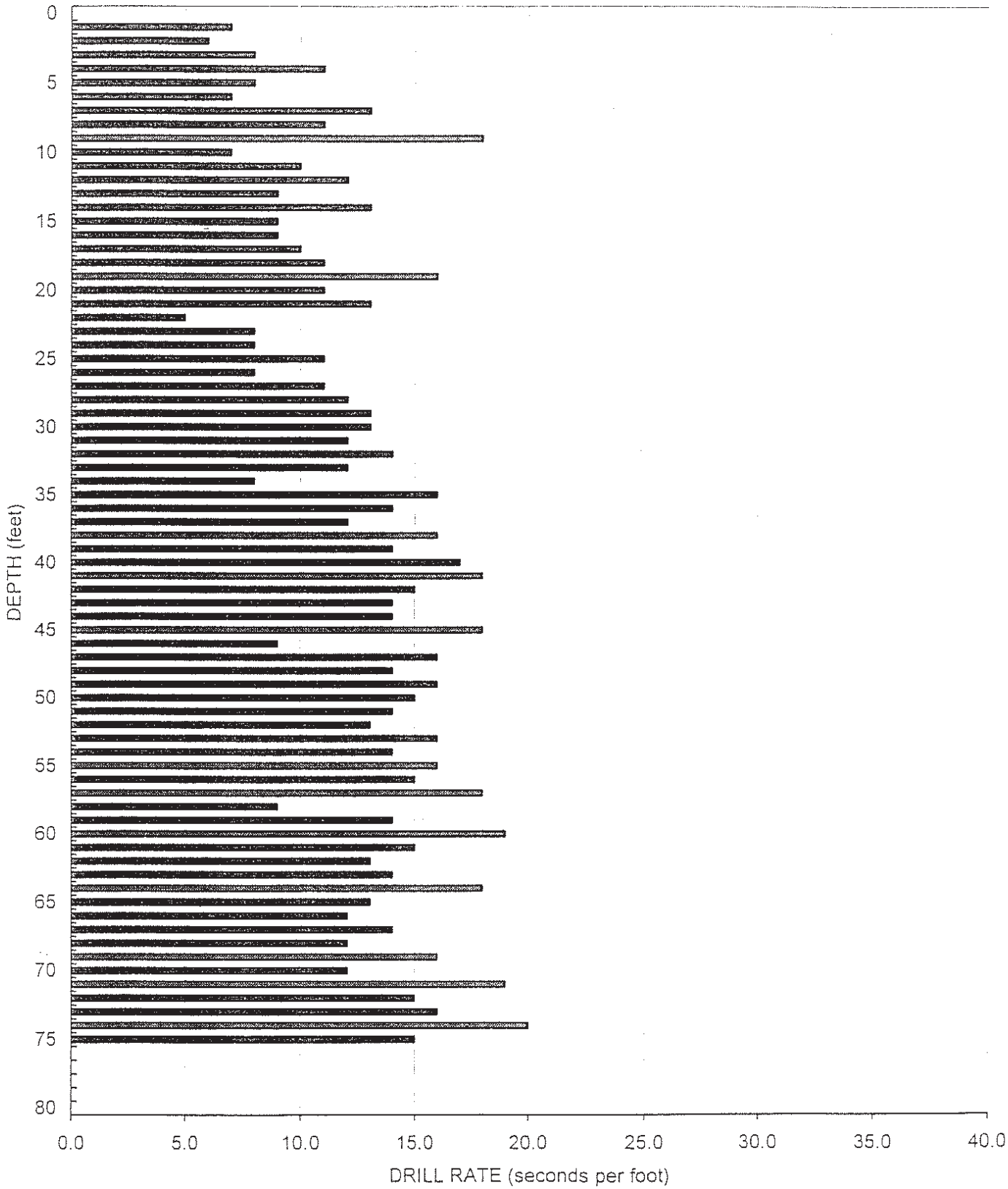
TRACT NO. 26372

AIR TRACK BORING AT-6

Elevation - 1660 Feet (MSL)

Date 07-02-2003 - Equipment: IR 370

GEOCON INCORPORATED



AT06.xls

FIGURE A-6

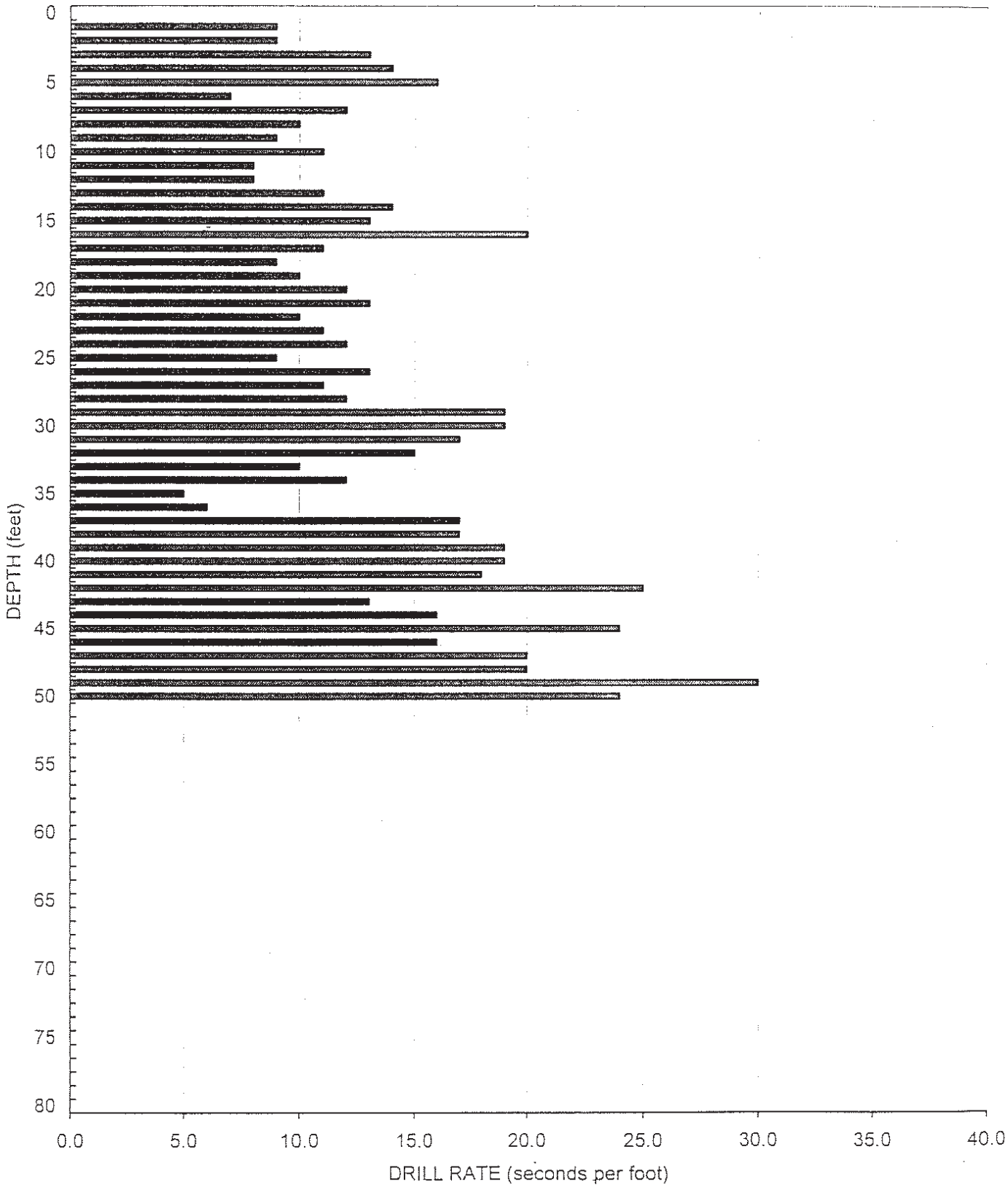
TRACT NO. 26372

AIR TRACK BORING AT-7

Elevation - 1654 Feet (MSL)

Date 07-02-2003 - Equipment: IR 370

GEOCON INCORPORATED



AT07.xls

FIGURE A-7

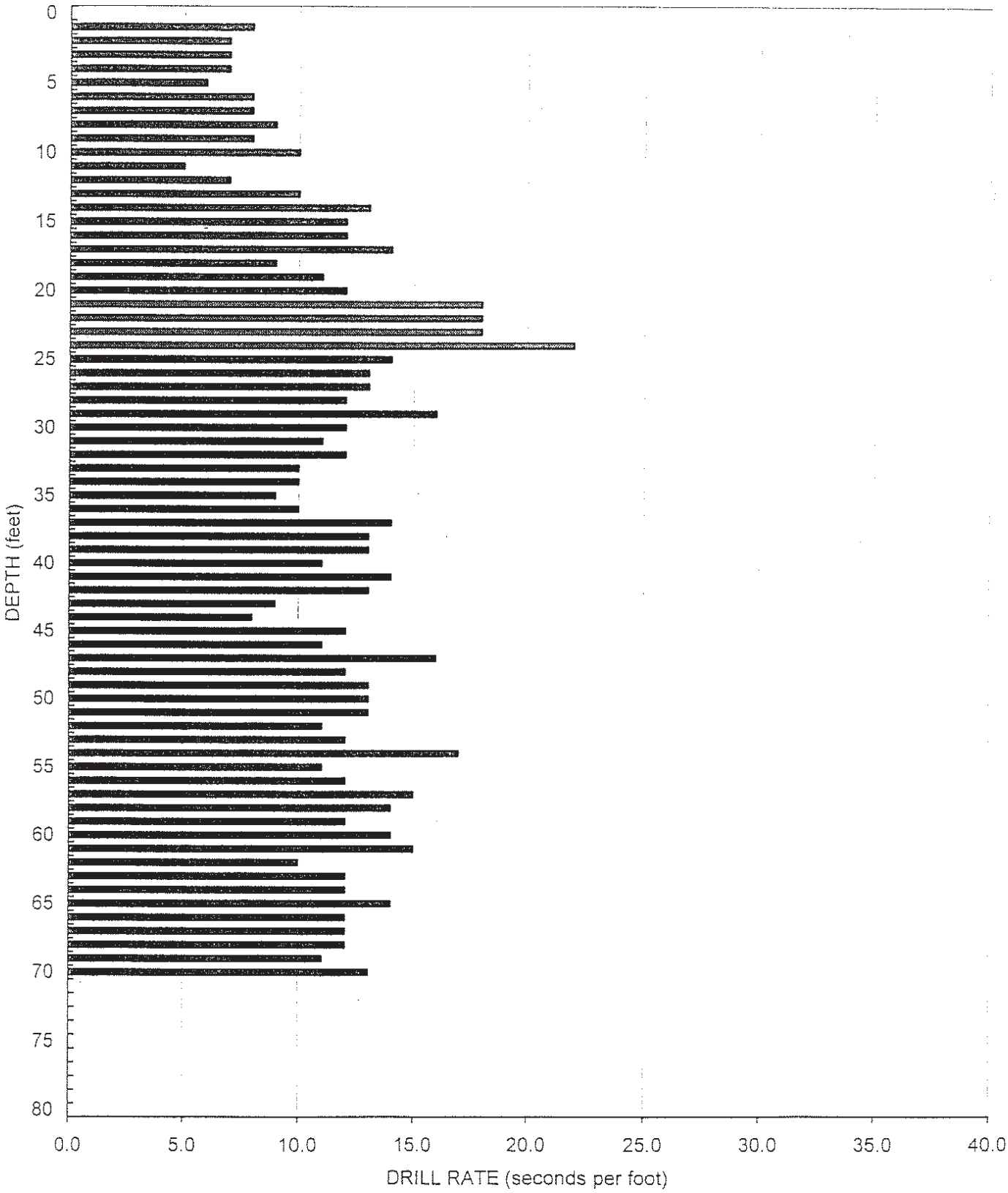
TRACT NO. 26372

AIR TRACK BORING AT-8

Elevation - 1656 Feet (MSL)

Date 07-02-2003 - Equipment: IR 370

GEOCON INCORPORATED



AT08.xls

FIGURE A-8

FIGURE C-5

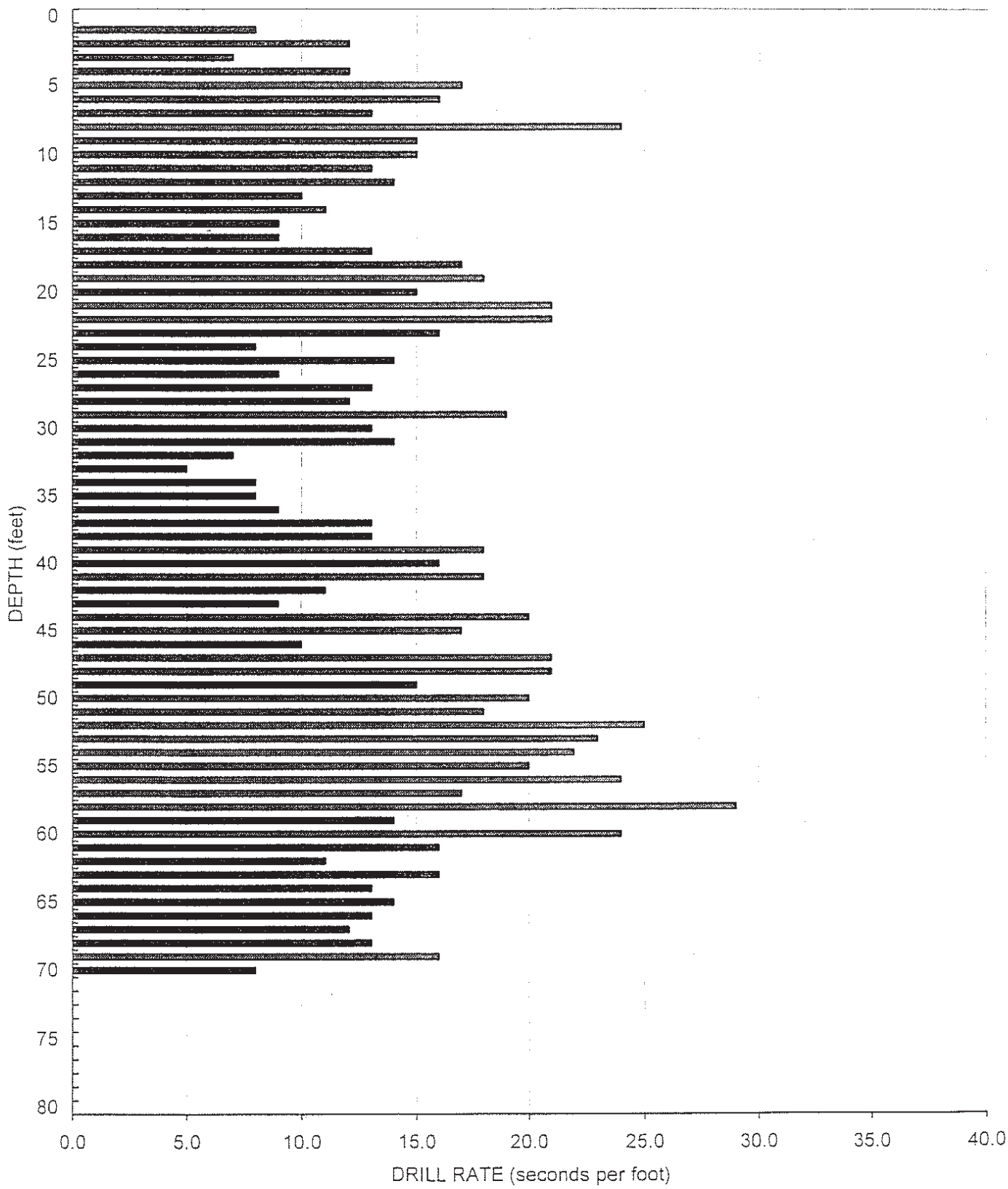
TRACT NO. 26372

AIR TRACK BORING AT-9

Elevation - 1686 Feet (MSL)

Date 07-02-2003 - Equipment: IR 370

GEOCON INCORPORATED



AT09.xls

FIGURE A-9

FIGURE C-6

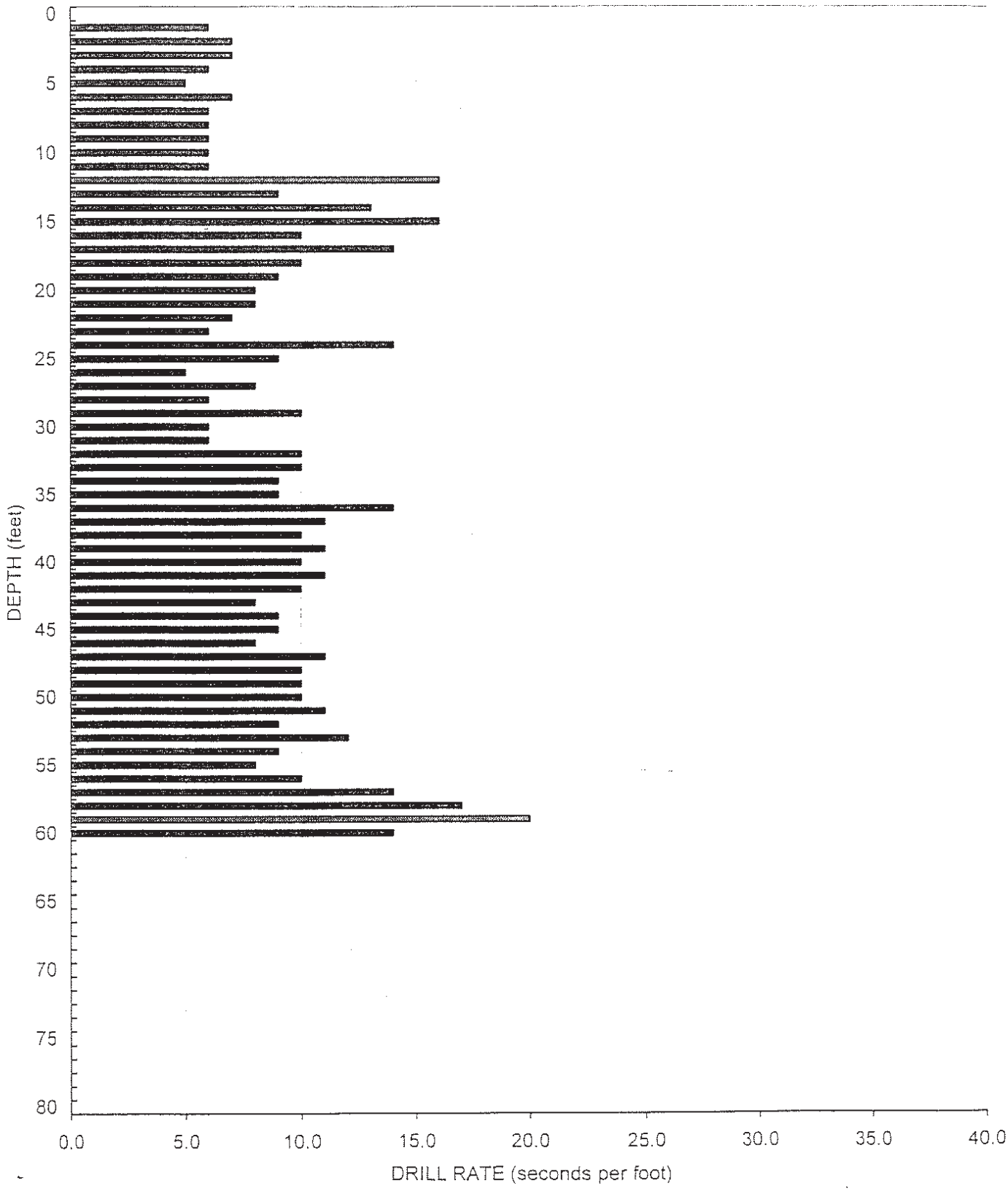
TRACT NO. 26372

AIR TRACK BORING AT-10

Elevation - 1660 Feet (MSL)

Date 07-02-2003 - Equipment: IR 370

GEOCON INCORPORATED



AT10.xls

FIGURE A-10

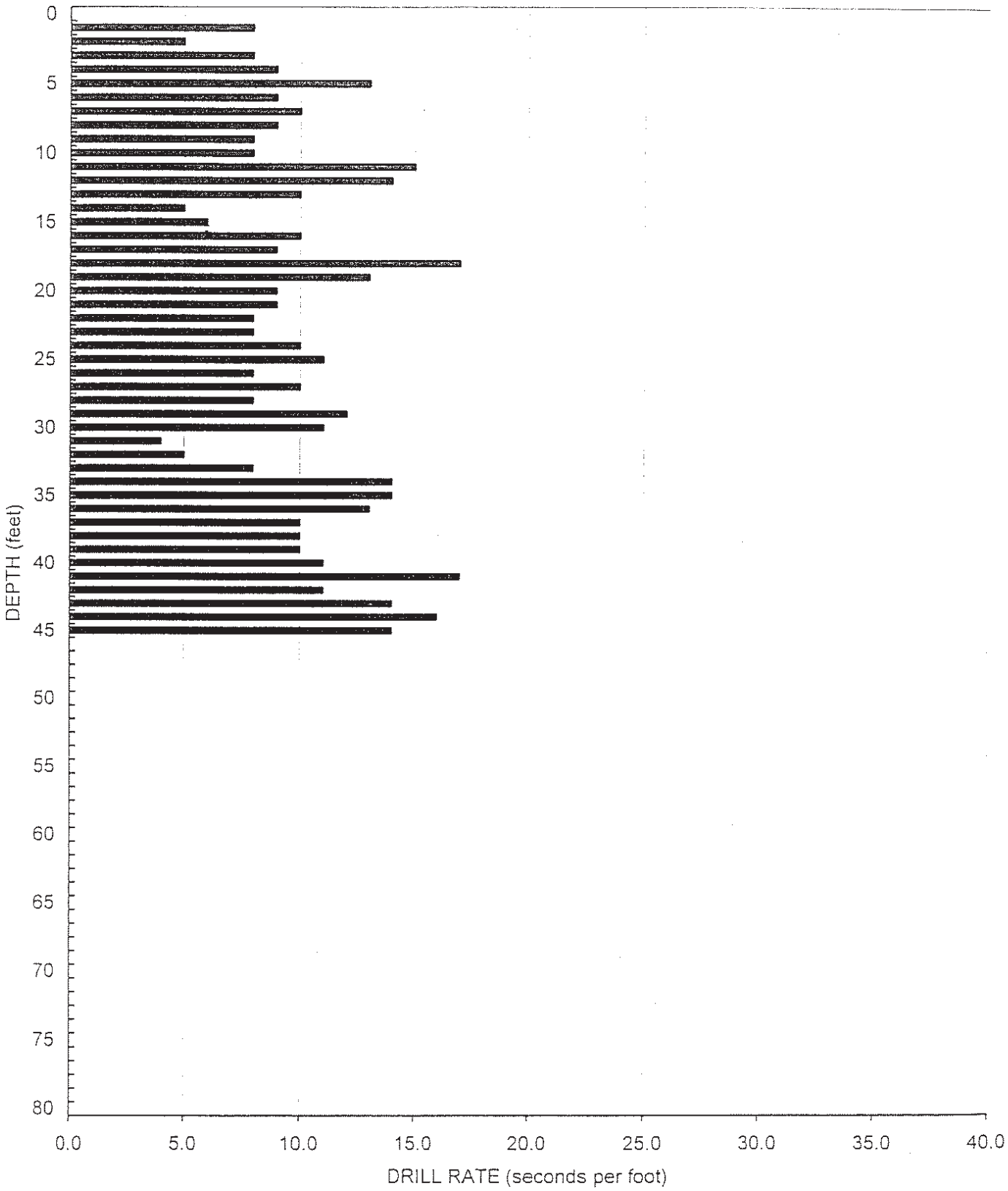
TRACT NO. 26372

AIR TRACK BORING AT-11

Elevation - 1640 Feet (MSL)

Date 07-02-2003 - Equipment: IR 370

GEOCON INCORPORATED



AT11.xls

FIGURE A-11

FIGURE C-8

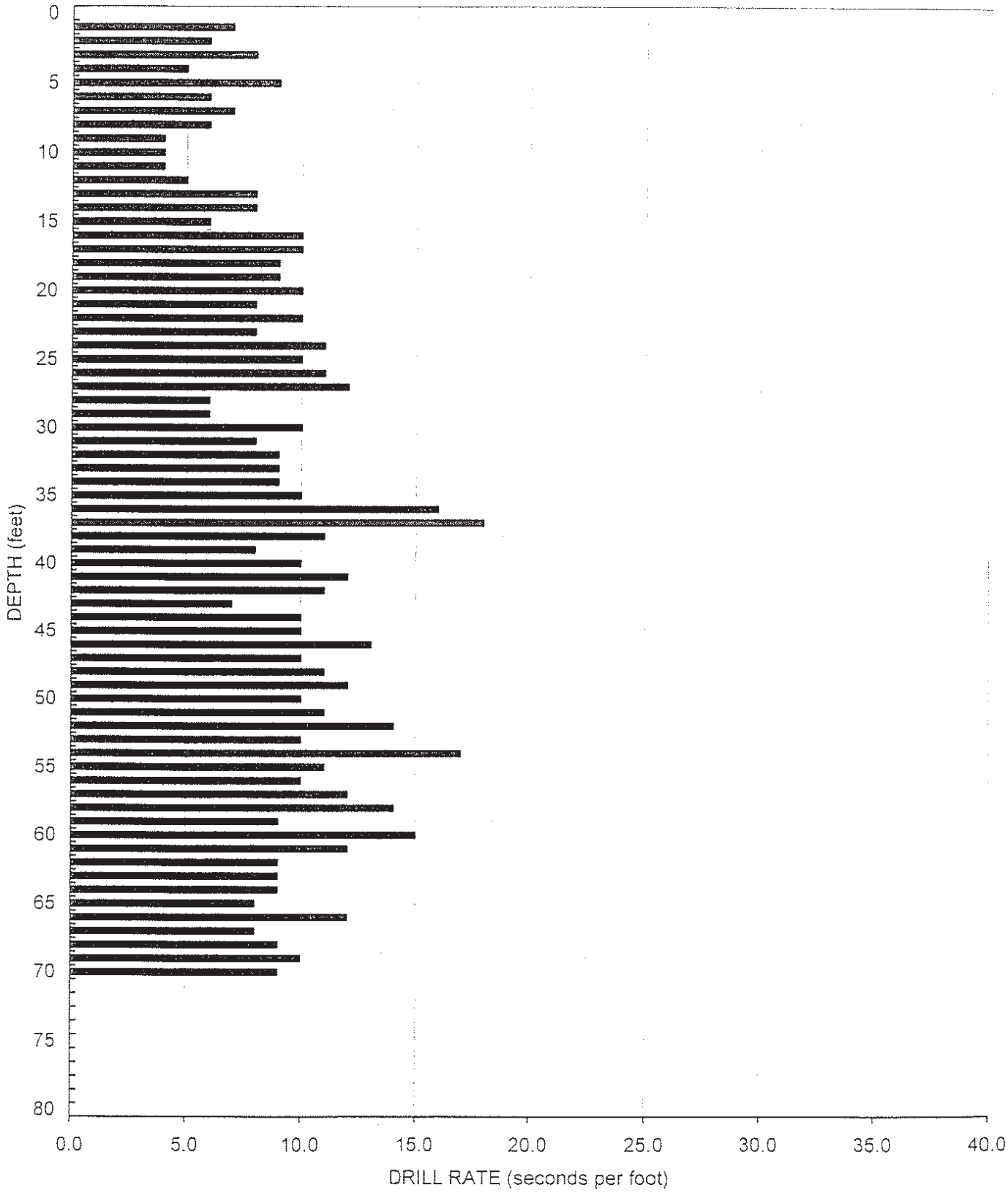
TRACT NO. 26372

AIR TRACK BORING AT-12

Elevation - 1654 Feet (MSL)

Date 07-02-2003 - Equipment: IR 370

GEOCON INCORPORATED



AT12.xls

FIGURE A-12

FIGURE C-9

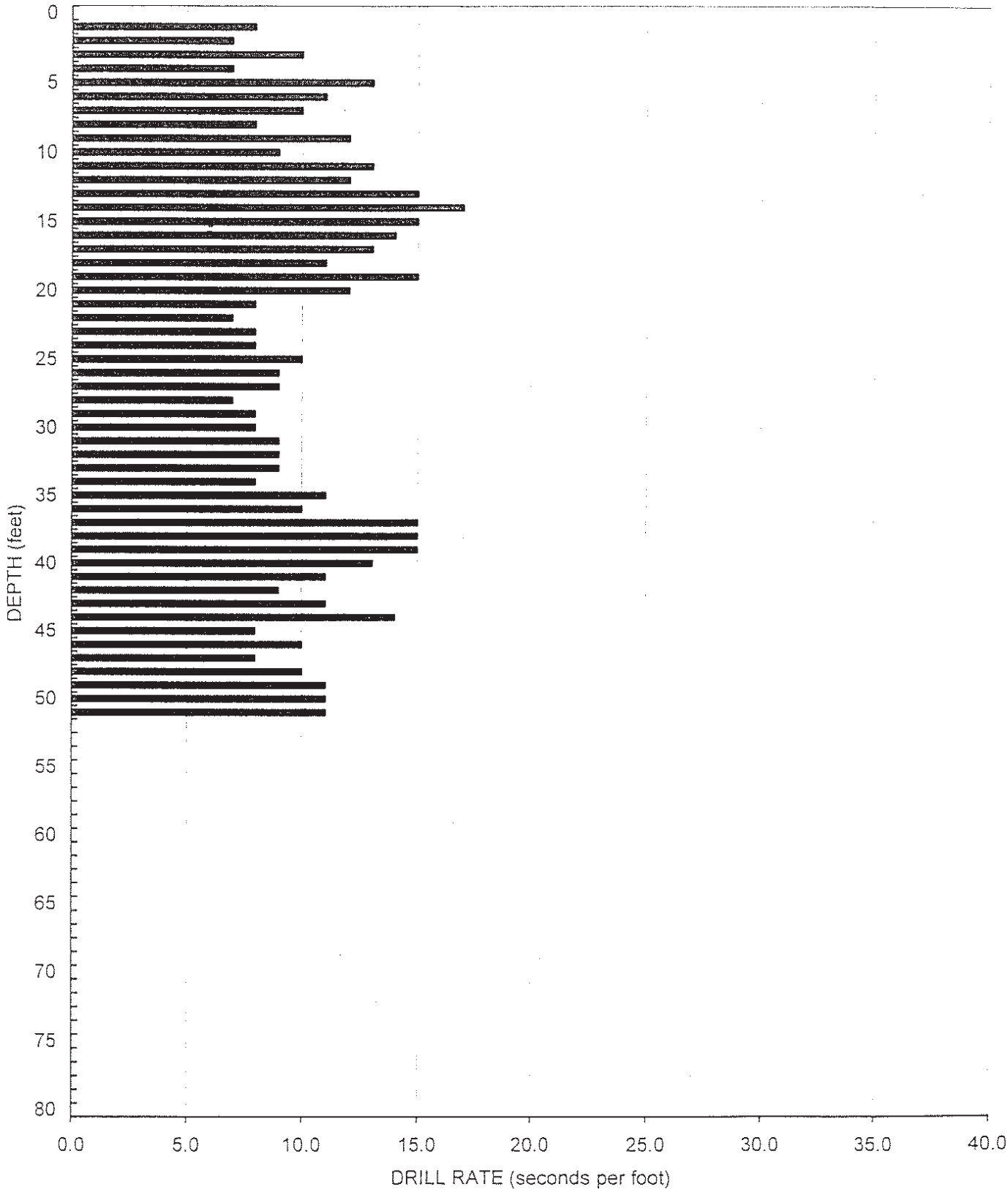
TRACT NO. 26372

AIR TRACK BORING AT-13

Elevation - 1640 Feet (MSL)

Date 07-02-2003 - Equipment: IR 370

GEOCON INCORPORATED



AT13.xls

FIGURE A-13

FIGURE C-10

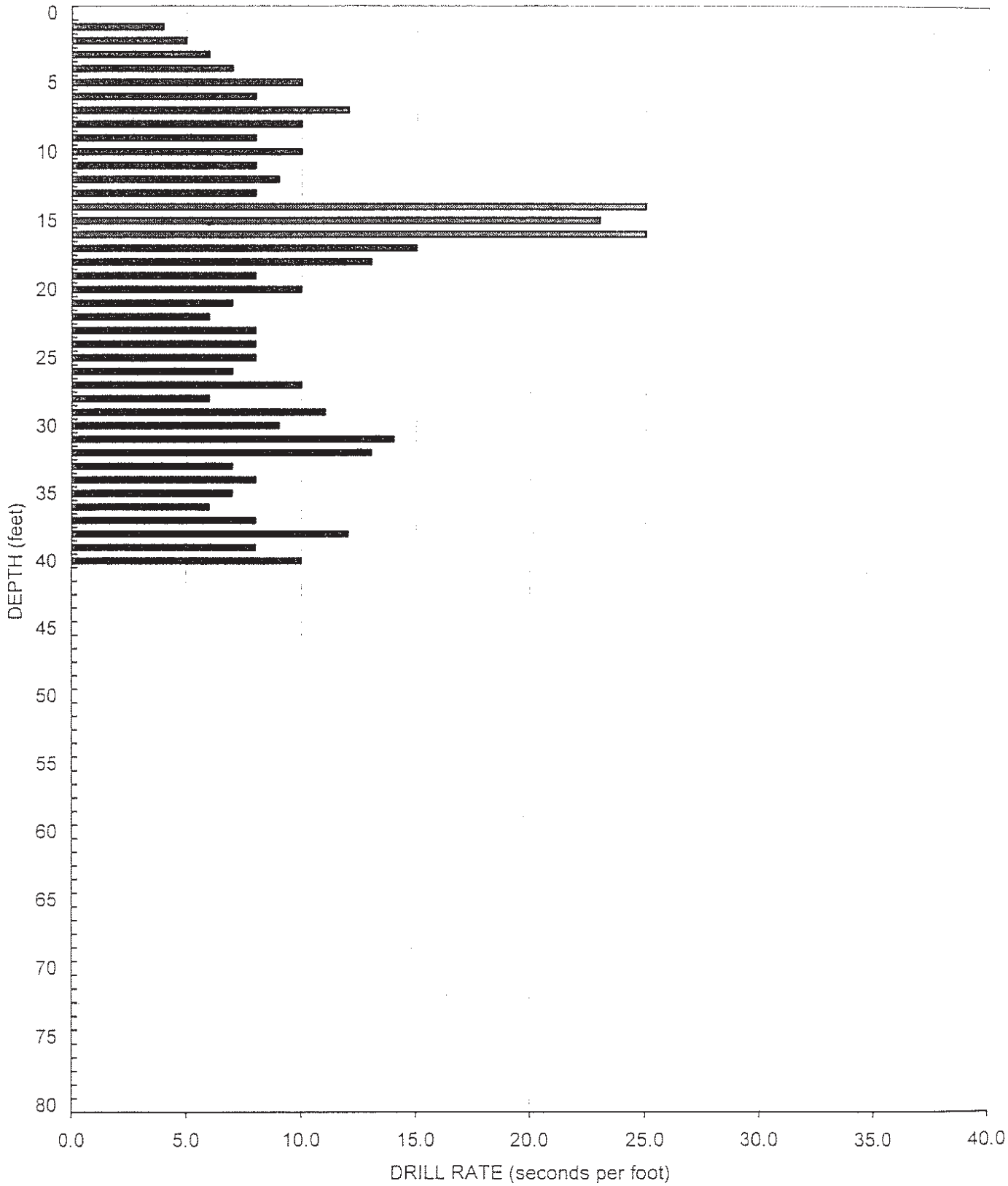
TRACT NO. 26372

AIR TRACK BORING AT-14

Elevation - 1620 Feet (MSL)

Date 07-02-2003 - Equipment: IR 370

GEOCON INCORPORATED



AT14.xls

FIGURE A-14

FIGURE C-11

PROJECT NO. 20145-12-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 1 ELEV. (MSL.) 1540 DATE COMPLETED 07/01/03 EQUIPMENT D8R DOZER	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	T1-1			SM	**TOPSOIL** Loose, medium to reddish brown, dry, Silty, fine to coarse SAND, trace gravel			
2				SM	**GRANITIC ROCK** Dense, highly weathered, medium brown to gray, fine to coarse-grained intrission igneous ROCK, moderately to highly fractured -Excavates as a Silty, fine to coarse SAND -Becomes highly to moderately weathered			
4								
6					TRENCH TERMINATED AT 6 FEET			

20145-12-01.GPJ

Figure A-15,
Log of Trench T 1, Page 1 of 1

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 20145-12-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 3 ELEV. (MSL.) 1566 DATE COMPLETED 07/01/03 EQUIPMENT D8R DOZER	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**TOPSOIL** Loose, medium to reddish brown, dry, Silty, fine to coarse SAND, trace gravel			
2				SM	**GRANITIC ROCK** Dense, highly to moderately weathered, gray, fine to coarse-grained intrussive igneous ROCK, moderately fractured -Excavates as a Silty, fine to coarse SAND			
4					-Becomes moderately to slightly weathered			
					TRENCH TERMINATED AT 5.5 FEET			

Figure A-17,
Log of Trench T 3, Page 1 of 1

20145-12-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 20145-12-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 4 ELEV. (MSL.) 1582 DATE COMPLETED 07/01/03 EQUIPMENT D8R DOZER	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	T4-1			SM	**TOPSOIL** Loose, reddish brown, dry, Silty, Clayey, fine to coarse SAND, trace fine gravel			
2 4				-SM	**GRANITIC ROCK** Dense, highly weathered, gray, fine to coarse-grained intrussive igneous ROCK, moderately fractured -Excavates as a Silty, fine to medium SAND			
					TRENCH TERMINATED AT 5 FEET			

Figure A-18,
Log of Trench T 4, Page 1 of 1

20145-12-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 20145-12-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 5 ELEV. (MSL.) _____ DATE COMPLETED 07/01/03 EQUIPMENT D8R DOZER	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**TOPSOIL** Loose, reddish brown, dry, Silty, fine to coarse SAND, trace clay, fine gravel			
2				SM				
	T5-1				**GRANITIC ROCK** Dense, highly weathered, gray, fine to coarse-grained intrussive igneous ROCK, moderately fractured			
4								
6					TRENCH TERMINATED AT 6 FEET			

Figure A-19,
Log of Trench T 5, Page 1 of 1

20145-12-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

APPENDIX B

LABORATORY TESTING

Laboratory tests were previously performed by Byerly (2001, 2002) on the easterly parcel (Tract No. 29426) in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Laboratory testing was also performed by Geocon for this investigation. Disturbed bulk representative samples collected during the Geocon investigation were also tested to determine maximum dry density and optimum moisture content, expansion characteristics and strength parameters. A water-soluble sulfate test was also conducted. Results of the laboratory tests are presented in tabular form herewith.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-00

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
TP-1*	Brown, silty fine to coarse SAND	132.6	7.2
TP-11*	Brown, silty fine to coarse SAND	130.4	9.3
B-2*	Light Brown silty fine to coarse SAND	133.4	9.7
T5-1	Brown to Grey silty fine to coarse SAND	130.2	9.5

*Data obtained from report by John R. Byerly Inc., dated March 14, 2002.

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-95

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
T5-1	8.3	22.1	116.6	25

TABLE B-III
SUMMARY OF DIRECT SHEAR TEST RESULTS

Sample No.	Sample Depth (ft)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
TP-2(1)	4.0	150	36
TP-3*(1)	3.5-12.5	175	42
TP-5*(1)	6.0-12.0	150	38
T5-1*	3.0-4.0	260	41

* Sample remolded to 90 percent relative compaction at near or slightly above optimum moisture content.

(1) Data obtained from report by John R. Byerly Inc., dated April 13, 2001.

TABLE B-IV
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	Water-Soluble Sulfate (%)	Sulfate Exposure
T5-1	0.015	Negligible

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

MULI-FAMILY DEVELOPMENT
BUNDY CANYON ROAD NEAR TULIP LANE
WILDOMAR, CALIFORNIA

PROJECT NO. T2145-22-02

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. 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, and/or 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

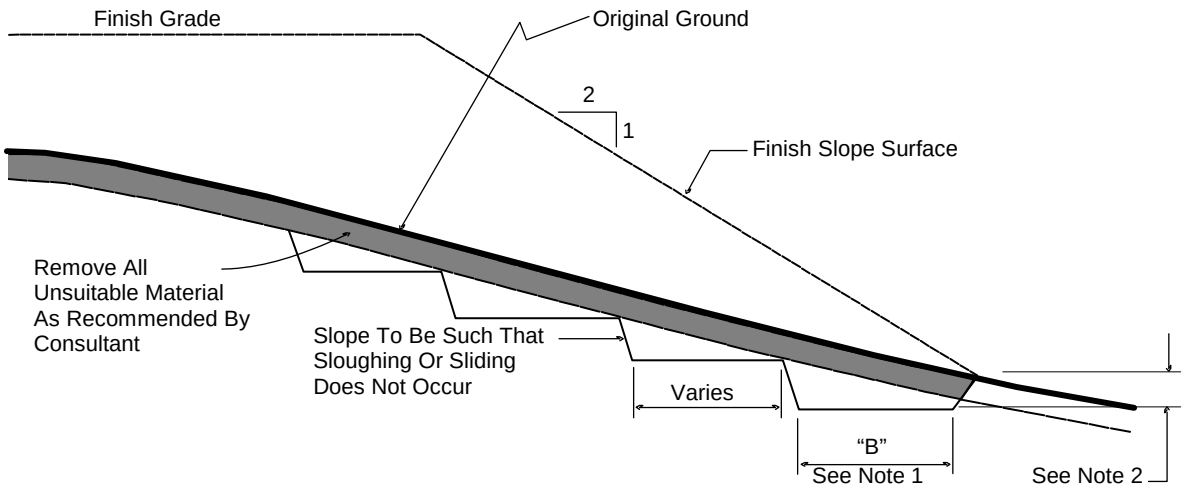
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material 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 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. 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



No Scale

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.

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. 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, 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.

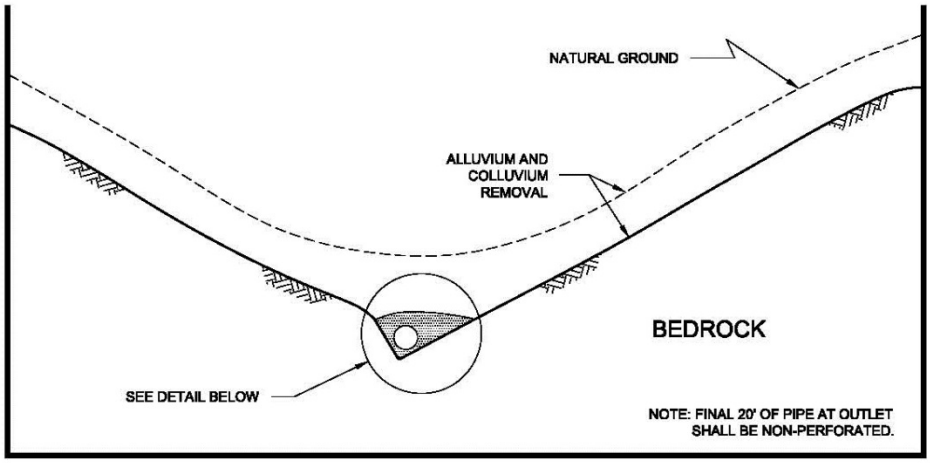
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant 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.

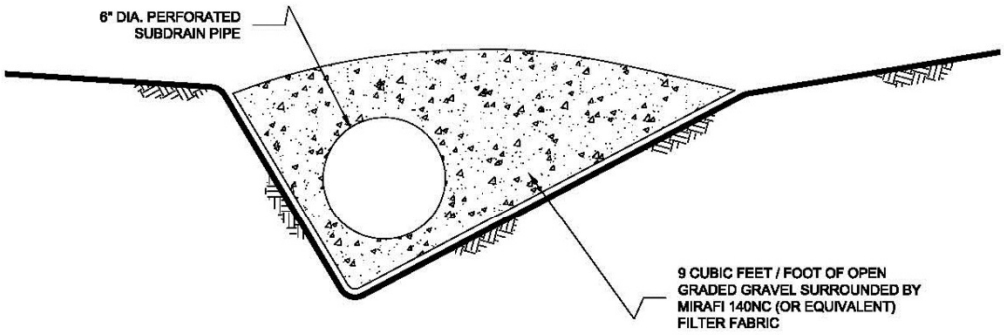
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

7.1 The geologic units on the site may have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to seepage. The use of canyon subdrains may be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Canyon subdrains with lengths in excess of 500 feet or extensions of existing offsite subdrains should use 8-inch-diameter pipes. Canyon subdrains less than 500 feet in length should use 6-inch-diameter pipes.

TYPICAL CANYON DRAIN DETAIL





NOTES:

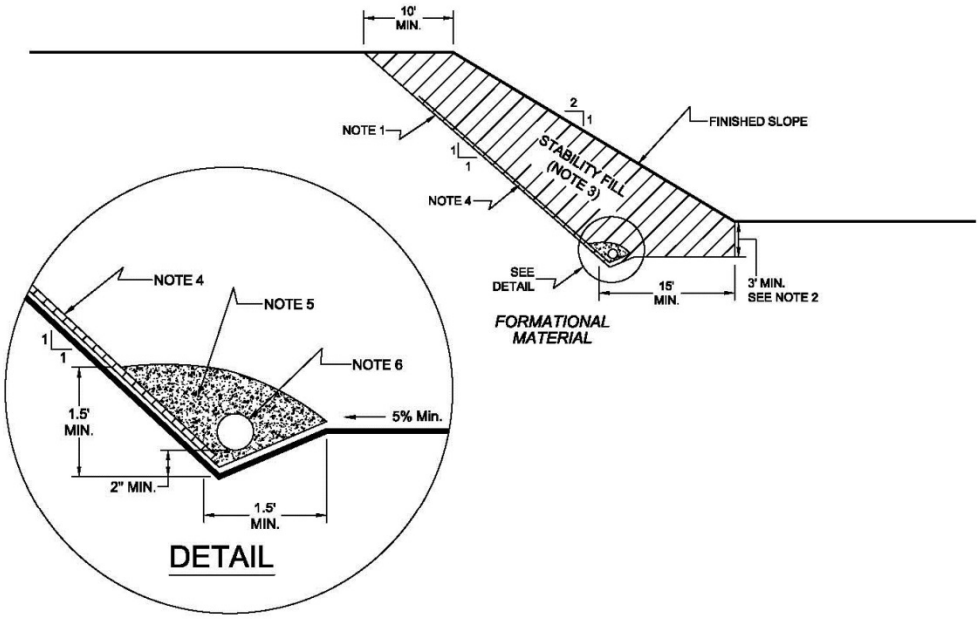
1.....8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS
IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

2.....6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS
LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 500 FEET.

NO SCALE

7.2 Slope drains within stability fill keyways should use 4-inch-diameter (or lager) pipes.

TYPICAL STABILITY FILL DETAIL



NOTES:

1.....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).

2.....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.

3.....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.

4.....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.

5.....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).

6.....COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET.

NO SCALE

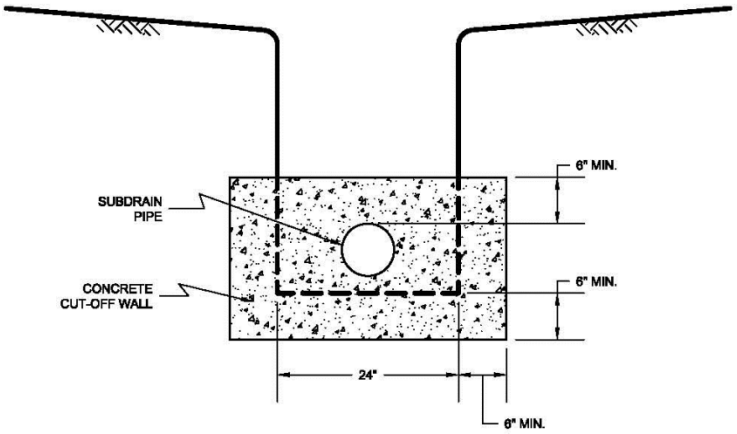
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing 40-scale grading plans.

7.4 *Rock* fill or *soil-rock* fill areas may require subdrains along their down-slope perimeters to mitigate the potential for buildup of water from construction or landscape irrigation. The subdrains should be at least 6-inch-diameter pipes encapsulated in gravel and filter fabric. *Rock* fill drains should be constructed using the same requirements as canyon subdrains.

7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

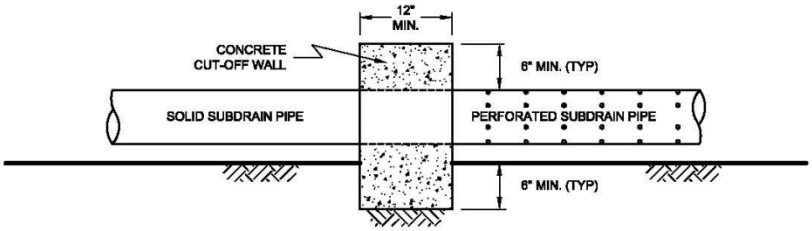
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

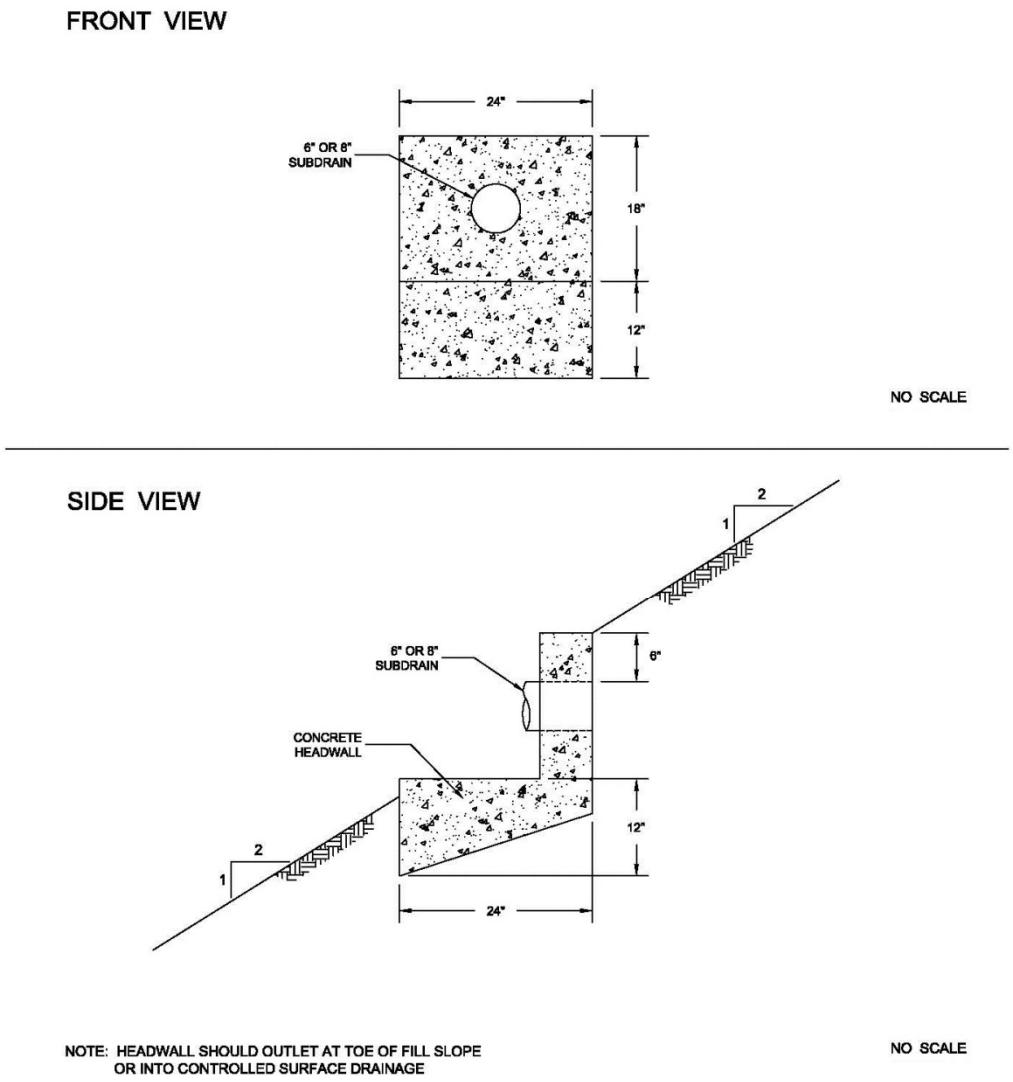
SIDE VIEW



NO SCALE

7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

TYPICAL HEADWALL DETAIL



7.7 The final grading plans should show the location of the proposed subdrains. After completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an “as-built” map showing the drain locations. The final outlet and connection locations should be determined during grading operations. Subdrains that will be extended on adjacent projects after grading can be placed on formational material and a vertical riser should be placed at the end of the subdrain. The grading contractor should consider videoing the subdrains shortly after burial to check proper installation and functionality. The contractor is responsible for the performance of the drains.

8. OBSERVATION AND TESTING

8.1 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.

8.2 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.

8.3 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.

8.4 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.

8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.

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

8.6.1 Soil and Soil-Rock Fills:

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

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

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

8.6.1.4. Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

9. PROTECTION OF WORK

9.1 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.

9.2 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.

10. CERTIFICATIONS AND FINAL REPORTS

10.1 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.

10.2 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.