Appendix F Geotechnical Report

NMG
Geotechnical, Inc.

January 31, 2012

Project No. 11117-01

To: TRG Land, Inc.
898 Production Place
Newport Beach, California 92663

Attention: Mr. Mark Rogers

Subject: Preliminary Geotechnical Exploration and Design Parameters for Proposed Single-Family Residential Development, Le Bard School Site, Huntington Beach, California

As requested, NMG Geotechnical, Inc. (NMG) has conducted a preliminary geotechnical exploration for the proposed single-family residential development planned at the subject site located north of Cynthia Drive, south of Crailet and Warwick Drive and east of residential units located along Suburbia Lane and Kenworth Court cul-de-sacs in the city of Huntington Beach, California. (Figure 1). The purpose of this exploration was to assess the onsite geotechnical conditions and provide preliminary recommendations for design, future grading and construction.

Our geotechnical exploration included advancement of two cone-penetration tests (CPT-1 and 2) to a depth of 50 feet, and drilling, sampling and logging of four hollow-stem auger borings (H-1, H-2, H-3 and H-4) to depths ranging from 31.5 to 51.5 feet. Laboratory testing was performed on selected soil samples to determine engineering soil properties.

The primary geotechnical constraints at the site are the presence of underlying compressible silty and clayey soils and shallow groundwater conditions. The laboratory test results indicate that the silty and clayey soils underlying the site are relatively compressible and historic high groundwater levels may be as shallow as 3 feet. Also, the site is mapped within a liquefaction zone and thin sand layers were identified below the groundwater table. The majority of the sandy soils are present at depths of below 25 feet. Based on our settlement, seismic and liquefaction analyses, static settlement on the order of 1 to 2 inches due to building and foundation loads, and seismic settlement on the order of 2¼ inches may occur during the design earthquake.

This report presents our findings, conclusions and preliminary recommendations for the proposed single-family residential development. Once the proposed development at the site is planned, additional geotechnical exploration, analysis and/or laboratory testing may be needed to further define the subsurface soil conditions and provide final recommendations for design, grading and construction specific to the development. Also, the future grading and foundation plans should be reviewed by the geotechnical consultant to confirm the actual design conditions and provide further recommendations, as needed.

If you have any questions regarding this report, please contact our office. We appreciate the opportunity to provide our services.

Respectfully submitted,

NMG GEOTECHNICAL, INC.

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1.0 INTRODUCTION

1.1 Site Conditions and Proposed Development

The subject site is roughly rectangular in shape and lies within the Orange County Coastal Plain at an elevation of approximately 8 to 10 feet Mean Sea Level (msl). The site is immediately west of the Santa Ana River and is separated by an existing levee. The subject site is bounded to the east by Cynthia Drive, west by Crailet and Warwick Drive and south by residential units located along Suburbia Lane and Kenworth Court cul-de-sacs in the city of Huntington Beach, California (Figure 1). The site is approximately 12.9 acres and consists of a city park, Huntington Beach City School District (HBCSD) headquarters and city of Huntington Beach Little League fields. Turf fields, as well as associated parking and concession buildings, are located along the western and southern portion of the site. A prior school, which now houses the HBCSD headquarters, is located within the northern and central portion of the site and a city park with play areas and tennis courts is located along the eastern edge, adjacent to the Southern California Edison (SCE) Easement. Residential structures are located along the northern, western and southern perimeters of the site. There are likely buried utilities at the site.

We understand the potential future development of the site will include construction of single family residential units. The project will include demolition of existing structures at the site and removal of extensive utilities. The actual future development, including the type of structures and layouts, is unknown at this time.

1.2 Scope of Services

Our scope of services for this study included the following tasks:

- Review of geotechnical information pertaining to the subject site, including site geology, historic groundwater data and seismic hazard maps (referenced in Appendix A).
- Site reconnaissance to identify the existing site conditions and marking of boring and CPT test locations.
- Notification of and coordination with Underground Service Alert and meeting with onsite representatives to identify and locate any underground utilities.
- Field exploration consisted of drilling, logging and sampling of four hollow-stem auger borings to depths of 31.5 to 51.5 feet and advancement of two CPTs to a depth of 50 feet. The boring and CPT logs are included in Appendix B.
- Laboratory testing was performed on selected samples to classify the onsite soils and evaluate in-situ moisture and density, maximum dry density and optimum moisture content, grain-size distribution, Atterberg limits, direct shear, consolidation, expansion index, R-value, and soil corrosivity (Appendix C).
- Preliminary geotechnical evaluation and analysis of the compiled data.
- Evaluation of faulting and seismicity in accordance with the 2010 California Building Code (CBC).

- Preparation of this report including our findings, conclusions, preliminary recommendations and accompanying illustrations.

NMG's expertise and scope of services do not include assessment of potential subsurface environmental contaminants or environmental health hazards.

1.3 Field Exploration

Our subsurface exploration was performed on December 22, 2011. Two CPTs (CPT-1 and CPT-2) were advanced to a depth of 50 feet. The CPTs were backfilled with bentonite chips. Four hollow-stem auger borings (Borings H-1, H-2, H-3 and H-4) were drilled to depths of approximately 31.5 to 51.5 feet below existing surface. The borings were geotechnically logged, and samples were obtained at selected intervals. Borings were backfilled with native soil cuttings. The approximate locations of the geotechnical borings and CPTs are shown on Figure 2 and the associated logs are provided in Appendix B.

We sampled the soils in the borings using a Modified California ring sampler (2.5-inch, inside-diameter, split-barrel). The sampler was driven with a 140-pound automatic hammer, free-falling 30 inches. We collected undisturbed ring samples from the borings at 2.5- to 5-foot intervals. Representative bulk samples of onsite soils were collected from the hollow-stem cuttings and were used for additional soil identification purposes and laboratory testing. The sampling was used to assess the soil beneath the site, as well as to obtain a measure of resistance of the soil to penetration (recorded as blows-per-foot on the geotechnical boring logs).

1.4 Laboratory Testing

We performed laboratory testing on representative samples of onsite soils collected during our field exploration to characterize their engineering properties. Selected relatively undisturbed and bulk soil samples were tested for:

- Moisture content and dry density
- Grain-size distribution
- Atterberg limits
- Direct shear
- Maximum dry density and optimum moisture content
- Consolidation
- Expansion index
- R-value
- Corrosivity

Laboratory tests were conducted in general conformance with applicable American Society for Testing and Materials (ASTM) standard test methods. The laboratory test results are provided in Appendix C. In-situ moisture content and dry density data are included on the geotechnical boring logs (Appendix B).

2.0 GEOTECHNICAL FINDINGS

2.1 Geologic Setting

The subject site is located in the Orange County Coastal Plain within the peninsular range province of southern California. The site lies along the southern edge of the northwest plunging synclinal Los Angeles Basin, which includes up to 4,400 feet of unconsolidated Pleistocene age marine and non-marine deposits with up to 170 feet of unconsolidated alluvial deposits at the surface (CDMG, 1998). The site lies immediately west of the Santa Ana River.

2.2 Geotechnical Conditions

Our borings indicate that the underlying soils are comprised of native alluvial deposits. Based on mapping by the state (USGS, 2004), the native soils are described as young Alluvial Fan Deposits (Qyf) and Wash Deposits (Qw) associated with the Santa Ana River.

Our borings and CPTs generally found that the underlying material consist of inter-layered olive brown and gray silts and clays that are generally wet to saturated and soft to medium stiff. The in-situ testing indicates dry densities ranging from 69.1 to 100.2 pounds per cubic foot (pcf) with water content varying from 20.5 to 52.6 percent.

Grain-size distribution and plasticity (Atterberg Limits) tests were conducted on samples considered representative of the alluvial soil in the upper 50 feet. The samples tested were generally classified as sandy silts and clays, with fines contents ranging from 59 to 79 percent. One sample in Boring H-4 and at depth of 45 feet was classified as silty sand with fine content of 30 percent. The clayey silt samples collected from the upper 5 feet were found to have liquid limits of 31 and 34 percent and plastic limits of 23 and 29 (USCS classification for MH). The clayey silty bulk samples, collected in the upper 5 feet had maximum dry densities of 116 and 117.5 pcf at optimum moisture contents of 13 and 12.5 percent, respectively. Onsite silty soils in the upper 5 feet have low expansion potential (Expansion Index of 24 and 34). The R-value of the near surface soil sample was 24.

Direct shear testing was conducted on two undisturbed clayey silt samples, collected at depths of 2.5 and 5.0 feet, in order to evaluate the strength properties of the underlying materials. The results of the direct shear test indicate ultimate internal friction angles of 28 and 29 degrees with cohesions of 80 and 100 pounds per square foot (psf), respectively. The peak internal friction angles were 28 and 31 degrees at cohesions of 80 and 130 psf, respectively.

The consolidation test result shows that onsite soils have moderate to high settlement potential. The collapse and swell potential of the samples were negligible (on the order of 0.01 percent) upon the introduction of water at 1.6 ksf axial loads.

A representative soil sample of the near-surface soils was sent to an outside laboratory for corrosivity testing. This testing included pH, soil resistivity, sulfate content and chloride content. The soil corrosion findings are presented in Section 2.6.

2.3 Groundwater

Groundwater levels were encountered in onsite borings at depths of 22.5 to 38.3 feet at the completion of drilling. Boring H-3 was left open for a period of 8 hours; however, the boring caved to a depth of 5 feet. Based on the collected soil samples, soils below 5 feet are generally saturated. Based on the fine grained nature of the onsite soils as well as the saturated conditions below 5 feet, the actual groundwater levels at the site may be significantly higher than was observed within the borings.

Based on the City of Huntington Beach General Plan (1996), mapping by the State, and prior groundwater data in the vicinity of the site, the historic high groundwater levels may be as shallow as 3 feet deep (CDMG, 1998). We believe the groundwater levels at the site fluctuate on an annual and seasonal basis; however, the groundwater is anticipated to be shallower than 10 feet and it may be encountered during grading and excavations deeper than 3 feet.

2.4 Regional Faulting and Seismicity

Regional Faults: The site is not located within a fault-rupture hazard zone as defined by the Alquist-Priolo Special Studies Zones Act (CDMG, 1999) and no evidence of active faulting was observed during this exploration. Also, based on mapping by the State (CDMG, 1994 and 1998), there are no active faults mapped at the site.

Using the USGS computer program (USGS, 2002, updated 2008) and the site coordinates of 33.6652 degrees north latitude and 117.9511 degrees west longitude, the closest major active faults to the site are the Newport-Inglewood Fault located 2.2 km south of the site and the San Joaquin Hill Blind Thrust located 3.3 km east of the site.

Seismicity: Sites in southern California are subject to seismic hazards of varying degrees depending upon the proximity, degree of activity, and capability of nearby faults. These hazards can be primary (i.e., directly related to the energy release of an earthquake such as surface rupture and ground shaking) or secondary (i.e., related to the effect of earthquake energy on the physical world, which can cause phenomena such as liquefaction and ground lurching). Since there are no active faults at the site, the potential for primary ground rupture is considered low. The primary seismic hazard for this site is ground shaking due to a future earthquake on one of the major regional active faults listed above.

The maximum moment magnitude for the controlling fault is 6.97 M_W, with peak ground accelerations of 0.46g (SDS/2.5) which would be generated from the San Joaquin Hills Blind thrust.

The site is located within an area of potential liquefaction, as defined by the State's Seismic Hazard Mapping Act. The attached Site Location and Seismic Hazards Map (Figure 1) shows the approximate location of the site relative to seismic hazard zones, as shown on the State of California Seismic Hazard Zones Map for the Los Newport Beach Quadrangle (CDMG, 1997). Liquefaction is discussed in Section 2.5.

Secondary seismic hazards, such as tsunami and seiche, are considered low as the site is located more than 2 miles away from the ocean and is not located within a mapped Tsunami Inundation Zone (CDMG, 2009).

2.5 Liquefaction Analysis

Liquefaction is a phenomenon in which earthquake-induced cyclic stresses generate excess pore-water pressure in low density (loose), saturated, sandy soils and soft silts below the water table. This causes a loss of shear strength and, in many cases, ground settlement. For liquefaction to occur, all of the following four conditions must be present:

- There must be severe ground shaking, such as occurs during a strong earthquake.
- The soil material must be saturated or nearly saturated, generally below the water table.
- The corrected normalized standard penetration test (SPT) blow counts (N_1) or the CPT tip resistance (Q) must be relatively low.
- The soil material must be granular (usually sands or silts) with, at most, only low plasticity. Clayey soils and silts of relatively high plasticity are generally not subject to liquefaction.

As previously discussed, the site is located within a mapped area of potential liquefaction (CDMG, 1997). Based on our seismic and liquefaction analysis and considering existing and historic high groundwater level (at depth of 3 feet), we estimate seismically induced settlement up to 2¼ inches during the design 6.97 M_w earthquake.

The thickness of the liquefiable sand layers varied from 1 to 2 feet thick and they are located predominately at depths below 10 feet with the exception of a layer of soil (2 feet thick) immediately below groundwater at depth of 3 to 5 feet below grades. This layer will need to be removed during the remedial grading at the site. The differential seismic settlement is not expected to be more than 1 inch over a span of 40 feet.

Since the soil in the upper 10 feet are generally fine grained with fine contents of more than 50 percent and due to relatively thin liquefiable layers below 10 feet, the potential for loss of bearing capacity in near surface soils is considered very low provided that the liquefiable layer in the upper 5 feet is removed and recompacted.

2.6 Corrosivity Testing

The corrosivity of one representative onsite soil sample was evaluated based electrical resistivity, pH, soluble sulfate and chloride content. The following table shows the test results:

Soil Corrosion Test	*Test Results*
Resistivity (ohm-cm) - Saturated	1,200
pH	8.4
Soluble Sulfate Content (ppm)	49.5
Chloride Content (ppm)	620

The electrical resistivity test on the saturated soil sample indicates that onsite soils are highly corrosive to ferrous metals. Sulfate-content test result indicates that onsite soils have "negligible" sulfate exposure per Table 4.3.1 of ACI-318. The chloride content is greater than 500 ppm which indicates corrosive conditions. The corrosivity test results are presented in Appendix C.

3.0 CONCLUSION AND RECOMMENDATIONS

3.1 General Conclusion and Recommendation

Based on our study, the proposed development of the subject site is considered feasible from a geotechnical standpoint provided the recommendations in this report and future geotechnical design reports are implemented during design, grading and construction. The site will require demolition and removal of the existing structures, as well as remedial grading to remove unsuitable soils and provide a compacted fill blanket to support the future development. Shallow groundwater should be anticipated and groundwater monitoring wells should be installed prior to grading.

Our recommendations are based on the anticipated geotechnical conditions and should be verified during grading and construction. Additional soil testing and revised recommendations may be necessary if exposed geotechnical conditions vary significantly from the findings and interpretations presented in this report. Geotechnical observation and testing should be conducted during demolition, grading and construction operations. The recommendations in this report are considered minimum and may be superseded by more stringent requirements of others and/or the future geotechnical consultant of record.

3.2 Site Preparation and Earthwork

Site preparation and grading should be performed in accordance with the recommendations herein and the requirements of the City of Huntington Beach. NMG's General Earthwork and Grading Specifications are included in Appendix E.

3.2.1 Site Demolition and Clearing

Prior to remedial grading and after demolition and removal of the existing structures, deleterious materials and debris should be cleared from the site and disposed of offsite. Excavation for the removal of existing foundations, utilities and vegetation should be observed by the geotechnical consultant. Large roots, highly organic soils, pipelines and any construction debris should be removed and should not be incorporated into new fills.

Soil that is disturbed as part of large excavations or removal of underground utilities should be observed and evaluated by the geotechnical consultant. Excavations that require backfill should be properly documented and compacted under the observation and testing of the geotechnical consultant.

3.2.2 Protection of Existing Improvements and Utilities

Existing buildings, improvements and utilities adjacent to the site that are to be protected in place should be located and visually marked prior to demolition and grading operations. Excavations adjacent to improvements to be protected in-place or any utility easement should be performed with care, so as not to undermine existing foundations or destabilize the adjacent ground.

Stockpiling of soils (more than 5 feet in height) at or near existing structures and over utility lines should not be allowed without review by the geotechnical consultant. If deeper removals are required, shoring or other special measures (i.e., setback or laybacks) for safety and to mitigate the potential for lateral/vertical soil movements may be required.

3.2.3 Remedial Grading Measures

The near-surface soils are considered unsuitable for structural support in the current condition. These materials should be removed and recompacted (per Section 3.2.4). The estimated remedial removals for the site is on the order of 5 feet deep to fully remove the soft and loose artificial fill and weathered alluvium. This removal would reduce the future settlement potential. The removal bottoms should be reviewed and approved by the geotechnical consultant prior to fill placement.

Since the majority of the remedial removals are relatively shallow (5 feet), they may consist of near vertical excavation and limited laybacks. The geotechnical consultant should observe the removal excavations and may provide additional recommendations based on the exposed conditions in the field.

Due to shallow groundwater conditions, soft, wet soils may be encountered at the bottom of the excavation. Select gravel or crushed rock materials may be required to provide a stabilized excavation bottom. Typically, one to two feet of gravel/crushed rock has been considered adequate for stabilization of the excavation bottoms. However, based on our experience with similar projects, stabilization of the excavation bottoms using a layer of geotextile material with gravel may be more viable.

It is imperative that once the removal bottom is achieved, heavy equipment not be allowed to traverse the area until the bottom is stabilized sufficiently to support the equipment without significant pumping or rutting.

3.2.4 Fill Placement

Upon completion of remedial removals, the approved removal bottoms should be scarified a minimum of 6 inches, except when soft, wet soils are encountered as indicated in Section 3.2.3. The removal bottoms and fill materials should be compacted to at least 90 percent of maximum dry density, as determined by ASTM Test Method D1557. Fill materials should be placed in loose lifts no thicker than 6 inches.

Fill materials should be relatively free of deleterious material. The existing fill soil and alluvium at the site should generally be suitable for re-use as compacted fill. The moisture content of new compacted fill soils can be placed at above the optimum moisture content within the compactable moisture range. Appropriate equipment support and other measures (e.g., mixing, stockpiling, drying) may be needed to achieve the uniform and correct moisture content for placement of the fill. If the soils become extremely wet, special measures for mixing and drying may be required that will need to be determined based on the field conditions.

3.2.5 Earthwork Shrinkage and Bulking

Due to the inherent variability of soil materials, earthwork volume changes are difficult to accurately quantify. Based on the gathered data and our experience with similar materials, we anticipate the weathered alluvium at the site to shrink between approximately 5 to 15 percent.

3.3 Seismic Design Parameters

The seismic design criteria based on the 2010 California Building Code (CBC) is as follows:

Selected Seismic Design Parameters from 2010 CBC	*Seismic Design Values*	*Reference*
Latitude	33.8137 North	
Longitude	118.0534 West	
Controlling Seismic Source	San Joaquin Hill Thrust	USGS, 2008
Distance to the Controlling Seismic Source	2.1 Miles (3.3 km)	USGS, 2008
Site Class per Table 1613.5.2	D	USGS, 2011
Spectral Acceleration for Short Periods (Ss)	1.737 g	USGS, 2011
Spectral Accelerations for 1-Second Periods (S1)	0.0.637 g	USGS, 2011
Five-percent damped Design Spectral Response Acceleration at Short Periods (S_{DS}) from Equation 16-39 (Site Class D)	1.158 g	USGS, 2011
Five-Percent Damped Design Spectral Response Acceleration at 1-Second Period (S_{D1}) from Equation 16-40 (Site Class D)	0.637 g	USGS, 2011

3.4 Foundations Design

Expansive soil conditions are expected to govern foundation and slab-on-grade design from a geotechnical standpoint. Foundation and slabs on expansive soils (EI>20) should be designed per the requirements of Section 1808.6 of California Building Code (CBC), 2010. The preliminary design parameters for post-tensioned and wire-reinforced slabs are provided below; however, these parameters may need to be revised upon further subsurface exploration and laboratory testing.

Shallow and mat foundations are suitable at the site when supported on compacted fill. The design of slab and foundation is the purview of the project structural engineer based on the anticipated dead and live loads. The design of foundations should also consider the settlement for static and seismic conditions as discussed in Section 3.6. In order to help limit the impacts of differential settlement induced by liquefaction and seismic shaking mat foundations or continuous footings may be required for the future structures. It should be noted that structures founded on isolated footings are generally prone to more differential settlement.

The fill under a mat foundation should be uniform, moisture-conditioned to above optimum-moisture content and compacted to a minimum relative compaction of 90 percent per ASTM 1557.

For preliminary design purposes, the net allowable bearing capacity for footings may be calculated based on the following equation:

$$q_{all} = 600\ D + 200\ B + 500$$

where:

D = embedment depth of footing, in feet
B = width of footing, in feet
q_{all} = maximum allowable bearing pressure, not to exceed 2,500 psf.

The allowable bearing pressure may be increased by one-third for wind and seismic loading. The coefficient of resistance of 0.35 against sliding is considered appropriate. For isolated footings, we recommend minimum embedment of 18 inches below lowest adjacent grade.

GEOTECHNICAL GUIDELINES
FOR DESIGN OF POST-TENSIONED SLABS*

Parameter	*Recommendation*
Center Lift	
* Edge Moisture Variation Distance, e_m	9.0 feet
* Center Lift, y_m	0.31 inches
Edge Lift	
* Edge Moisture Variation Distance, e_m	4.60 feet
* Edge Lift, y_m	0.41 inch
Effective Plasticity Index (PI)	8
Subgrade Modulus, k	75 pci
Modulus of Elasticity of Soils, Es	1,000 psi
Presaturation, as needed, to obtain the minimum moisture down to the minimum depth	1.2 x optimum down to 12 inches
****Based on method in CBC 2010***	

For uniform thickness post-tensioned slabs, we recommend that the slabs have a thickened edge such that the slab is embedded a minimum of 12 inches below the lowest adjacent grade. The thickened edge should be tapered and have a minimum width of 12 inches. If non-uniform (ribbed) post-tensioned slabs are used, we recommend a minimum embedment of 18 inches below adjacent grade for the thickened edges.

3.5 Interior Slab Moisture Mitigation

In addition to geotechnical and structural considerations, the project owner should also consider moisture mitigation when designing and constructing slabs-on-grade. The intended use of the interior space, type of flooring, and the type of goods in contact with the floor may dictate the need for, and design of, measures to mitigate potential effects of moisture emission from and/or moisture vapor transmission through the slab. A vapor retarder or barrier is typical under the slab to help mitigate moisture transmission through slabs.

Guidelines by the American Concrete Institute (ACI) (302.1R-96) recommend that the vapor retarder be placed directly under the slab (sand layer not required). However, the location of the vapor retarder and the use of sand above it may also be subject to the owner's/builder's past successful practice. A minimum 10-mil thick vapor retarder is recommended where flooring and/or interior use requires floor slab water vapor control.

Concrete mix design and curing are also significant factors in mitigating slab moisture problems. Concrete with lower water/cement ratios results in denser, less permeable slabs. They also "dry" faster with regard to when flooring can be installed (reduced moisture emissions quantities and rates). Rewetting of the slab following curing should be avoided since this can result in additional drying time required prior to flooring installation. Proper concrete slab testing prior to flooring installation is also important.

The concrete mix design and the type and location of the vapor retarder should be determined in coordination with all parties involved in the finished product, including the project owner, architect, structural engineer, geotechnical consultant, concrete subcontractors, and flooring subcontractors.

3.6 Static and Seismic Settlement Potential

Based on the consolidation testing, recommended remedial measures and anticipated foundation loads, the total consolidation (static) settlement for lightly loaded structures should be on the order of 1 to 2 inches. The differential settlement is typically half of the total settlement over a 40-foot span. The settlement at the site due to foundation and structural loads will need to be evaluated once additional explorations are performed at the site and updated for the actual structural loads.

The calculated settlement due to seismic shaking and liquefaction in the event of a strong ground shaking is on the order 2¼ inches as discussed previously. The differential seismic settlement is not expected to be more than 1 inch over a span of 40 feet. For structural design, the calculated static and seismic settlements do not need to be combined since the timing and mechanics of the settlements are very different.

3.7 Lateral Earth Pressures

The recommended lateral earth pressures based on our limited subsurface exploration and for approved compacted soils in drained conditions are as follows:

Conditions	*Level (pcf)*	*2:1 Slope (pcf)*
Active	45	75
At-Rest	65	95
Passive	330	120 (sloping down)

In addition to the above lateral forces due to retained earth, the influence of surcharge due to other loads such as adjacent footings, vehicular traffic or lateral loads acting on the retaining wall, if any, should be considered during the design of retaining walls. Recommendations for drainage behind retaining walls are provided in the attached detail (Figure 3, rear of text).

To design an unrestrained retaining structure, such as a cantilever wall, the active earth pressure may be used. For a restrained retaining structure, such as a basement wall, loading docks or at restrained-wall corners, the at-rest pressure should be used. Passive pressure is used to compute lateral soil resistance developed against lateral structural movement. Further, for sliding resistance, the friction coefficient of 0.35 may be used at the concrete and soil interface. In combining the total lateral resistance, either the passive pressure or the frictional resistance should be reduced by 50 percent. In addition, the passive resistance is taken into account only if it is ensured that the soil against embedded structures will remain intact with time. Drainage behind retaining walls should also be provided.

The seismic lateral earth pressure for level backfill may be estimated to be an additional 14 pcf for active and at-rest conditions. The earthquake soil pressure distribution is similar to active and at-rest pressure distributions and is added to the static pressures. For the active and at-rest conditions, the additional earthquake loading is zero at the top and maximum at the bottom.

3.8 Cement Type

As discussed before, our laboratory testing shows "negligible" levels of soluble sulfates in the collected soil sample. Additional laboratory testing will need to be performed during the future geotechnical exploration in order to better determine the soluble sulfate content of the onsite soils.

3.9 Soil Corrosivity

Based on our laboratory test results, the collected soil sample is highly corrosive to ferrous metals and the chloride content is greater than 500 ppm, which indicates corrosive conditions. Additional laboratory testing will need to be performed to determine the corrosion potential of onsite soils to ferrous metals. If required, specific corrosion protection recommendations for buried iron/steel pipes and structural elements should be provided by a corrosion engineer.

3.10 Asphalt Concrete Pavement

As discussed previously, the R-value of the collected near surface soil sample was 24. For preliminary purpose using a traffic index (TI) of 4.0 for parking stalls, and TI of 5.5 for drive areas, and a design R-value of 20, we recommend the following pavement sections in accordance with the Orange County Highway Design Manual. Please note that City of Huntington Beach requires minimum pavement section consisting of 4 inches of asphalt concrete (AC) over 6 inches of aggregate base (AB) for residential developments and 4.8 inches of AC over 10 inches of AB for commercial and industrial developments.

T.I. = 4.0	*T.I. = 5.5*
	7-inch Full Depth AC
4-inch AC/6-inch AB	4-inch AC/7-inch AB

Additional sampling and testing at or near the completion of street grading should be performed in order to provide final structural pavement recommendations. Also, the traffic index of the potential streets and parking lots within the site will need to be determined by a traffic engineer prior to finalization of the pavement sections.

Pavement sections should be placed in accordance with the requirements of Section 301 and 302 of the Standard Specifications of Public Works Construction (The Green Book). Prior to construction of pavement sections, the subgrade soils should be scarified to a minimum depth of 6 inches, moisture-conditioned as needed, and recompacted in place to a minimum of 90 percent relative compaction per ASTM D1557. If AC is placed directly over the subgrade soil, then the subgrade soil needs to be compacted to a minimum relative compaction of 95 percent. Subgrade should be firm and unyielding.

AB materials should be crushed aggregate or crushed miscellaneous base in accordance with The Green Book. The materials should be free of any deleterious materials. AB materials should be placed in 6- to 8-inch loose lifts, moisture-conditioned as necessary, and compacted to a minimum of 95 percent relative compaction per ASTM D1557. AC should also be compacted to 95 percent relative compaction.

Moisture and root barriers should be considered along the street pavements that are adjacent to unpaved medians and parkways with landscape and irrigation in order to minimize the potential for wetting of the street subgrade soils and pavement distress.

3.11 Exterior Concrete

Based on our limited laboratory testing on the near surface soil sample taken at the subject site, the expansion index ranged from 24 to 34 which corresponds to "Low" expansion potential, in accordance with ASTM D4829 test method. Exterior concrete elements such as curb and gutter, driveways, sidewalks and patios are susceptible to lifting and cracking when constructed over expansive soils. With expansive soils, the impacts to flatwork/hardscape can be significant, generally requiring removal and replacement of the affected improvements. Please also note that

reducing concrete problems is often a function of proper slab design; concrete mix design, placement, and curing/finishing practices. Adherence to guidelines of the American Concrete Institute (ACI) is recommended. Also, the amount of post-construction watering, or lack thereof, can have a very significant impact on the adjacent concrete flatwork.

For reducing the potential effects of expansive soils, we recommend a combination of presaturation of subgrade soils; reinforcement; moisture barriers/drains; and a sub layer of granular material. Though these types of measures may not completely eliminate adverse impacts, application of these measures can significantly reduce the impacts from post-construction expansion of soil. The degrees and combinations of these measures will depend upon:

- The expansion potential of the subgrade soils;
- The potential for moisture migration to the subgrade;
- The feasibility of the measures (especially presaturation); and
- The economics of these measures versus the benefits.

These factors should be weighed by the project owner determining the measures to be applied on a project-by-project basis, subject to the requirements of the local building/grading department.

The following table provides our recommendations for varying expansion characteristics of subgrade soils. Additional considerations are also provided after the table. For preliminary purposes, we recommend that the **"low"** category be used during the preliminary design of the project site. Additional laboratory testing following the additional subsurface exploration and at the completion of grading operations should be performed to verify our preliminary recommendations.

TYPICAL RECOMMENDATIONS FOR CONCRETE FLATWORK/HARDSCAPE					
	Expansion Potential (Index)				
Recommendations	***Very Low (< 20)***	***Low (20 – 50)***	***Medium (51 – 90)***	***High (91 – 130)***	***Very High (> 130)***
Slab Thickness (Min.): Nominal thickness except where noted.	4"	**4"**	4"	4"	4" Full
Subbase: Thickness of sand or gravel layer below concrete	N/A	**N/A**	Optional	2" – 4"	2" – 4"
Presaturation: Degree of optimum moisture content (opt.) and depth of saturation	Pre-wet Only	**1.1 x opt. to 6"**	1.2 x opt. to 12"	1.3 x opt. to 18"	1.4 x opt. to 24"
Joints: Maximum spacing of control joints. Joint should be ¼ of total thickness	10'	**10'**	8'	6'	6'
Reinforcement: Rebar or equivalent welded wire mesh placed near mid-height of slab	N/A	**N/A**	Optional (WWF 6 x 6 – W1.4xW1.4)	No. 3 rebar, 24" O.C. both ways or equivalent wire mesh	No. 3 rebar, 24" O.C. both ways
Restraint: Slip dowels across cold joints; between sidewalk and curb	N/A	**N/A**	Optional	Across cold joints	Across cold joints (and into curb)

The more expansive soils, because they are clayey, can take significantly longer to achieve recommended presaturation levels. Therefore, the procedure and timing should be carefully planned in advance of construction. For exterior slabs, the use of a granular sublayer is primarily intended to facilitate presaturation and subsequent construction by providing a better working surface over the saturated soil. It also helps retain the added moisture in the native soil in the event that the slab is not placed immediately. Where these factors are not significant, the layer may be omitted.

On projects with highly expansive soils, additional measures such as thickened concrete edges/footings, subdrains and/or moisture barriers should be considered where planter or natural areas with irrigation are located adjacent to the concrete improvements. Design and maintenance of proper surface drainage is also very important. If the concrete will be subject to heavy loading from cars/trucks or other heavy objects, thicker slabs should be used.

The above recommendations typically are not applied to curb and gutter, but should be considered in areas with highly expansive soils.

3.12 Trench Excavation and Backfill

Excavations should be performed in accordance with the requirements set forth by Cal/OSHA Excavation Safety Regulations (Construction Safety Orders, Section 1504, 1539 through 1547,

Title 8, California Code of Regulations). In general, onsite soils are anticipated to be classified as Type "C". Cal/OSHA regulations apply to excavations that are up to 20 feet deep.

Trenches, including interior utility, should be either backfilled with native soil and compacted to 90 percent relative compaction, or backfilled with clean sand (SE 30 or better), which can be densified with water jetting and flooding. We recommend that backfill soils be placed at or above above optimum moisture content.

3.13 Drainage and Irrigation

Inadequate control of run-off water, heavy irrigation after development of the site, or regional groundwater level changes may result in shallow groundwater conditions where previously none existed. Maintaining adequate surface drainage, proper disposal of run-off water, and control of irrigation will help reduce the potential for future moisture-related problems and differential movements from soil heave/settlement.

Surface drainage should be carefully taken into consideration during grading, landscaping, and building construction. Positive surface drainage should be provided to direct surface water away from structures and slopes and toward the street or suitable drainage devices. Ponding of water adjacent to the structures should not be allowed. Paved areas should be provided with adequate drainage devices, gradients, and curbing to prevent run-off flowing from paved areas onto adjacent unpaved areas.

The performance of foundations is also dependent upon maintaining adequate surface drainage away from structures. The minimum gradient within 5 feet of the buildings will depend upon surface landscaping. In general, we suggest that unpaved lawn and landscape areas have a minimum gradient of 2 percent away from structures.

Construction of planter areas immediately adjacent to structures should be avoided. If planter boxes are constructed adjacent to or near buildings, the sides and bottoms of the planter should be provided with a moisture barrier to prevent penetration of the irrigation water into the subgrade. Provisions should be made to drain excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters. Raised planter boxes may be drained with weepholes. Deep planters (such as palm tree planters) should be drained with below-ground, water-tight drainage lines connected to a suitable outlet.

3.14 Future Geotechnical Exploration and Review of Plans

Additional geotechnical exploration may be needed for the project once the proposed development is planned. Future plans for the proposed site development and the grading plan should be reviewed and accepted by the geotechnical consultant. Additional recommendations or modifications to the recommendations herein may be necessary at that time. The geotechnical consultant should also review the foundation plans for conformance with the geotechnical design parameters and to determine the total and differential settlement for the structures at the site.

3.15 Observation and Testing during Grading and Construction

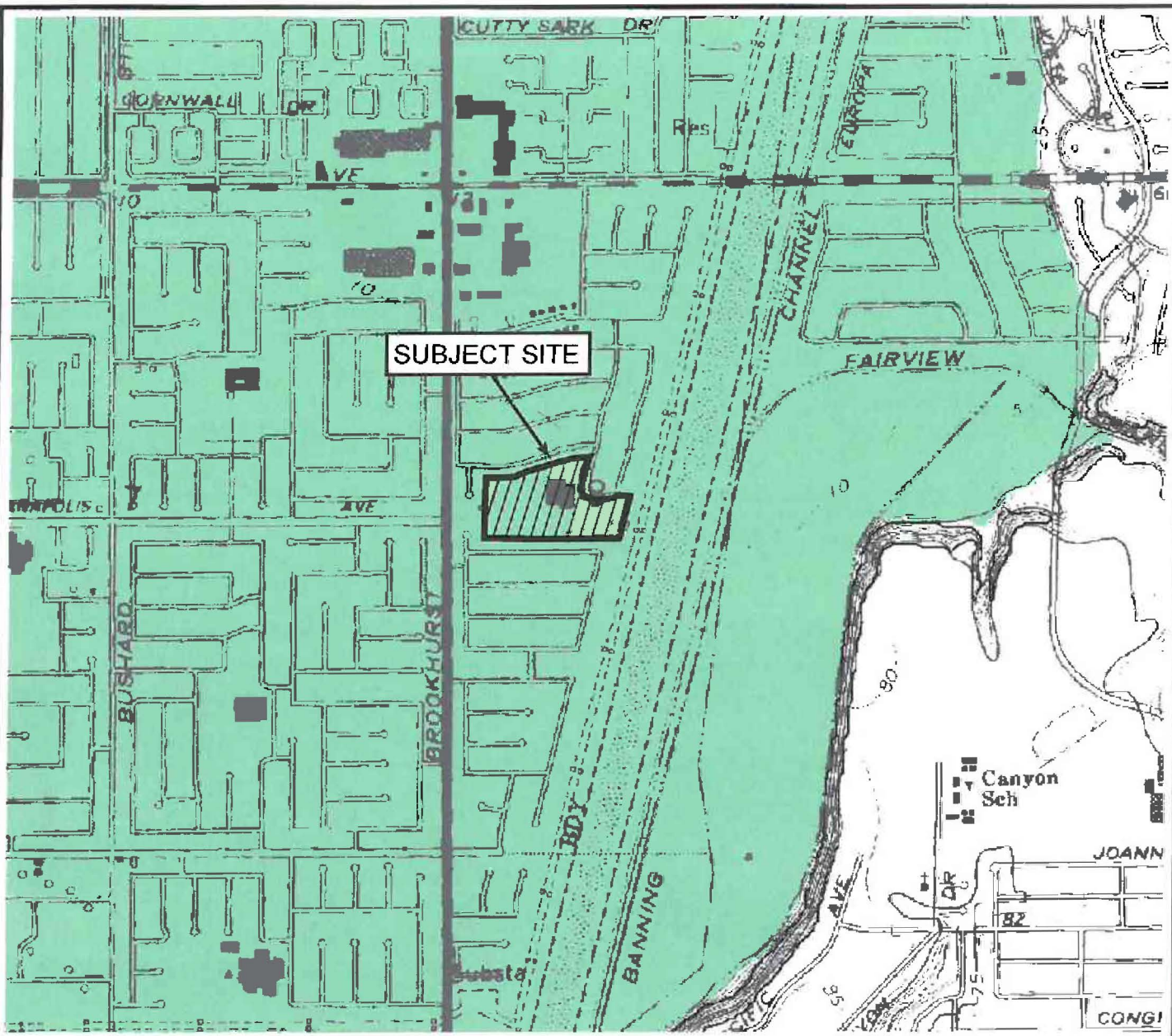
Geotechnical observation and testing should be performed by the geotechnical consultant of record during the following phases of grading and construction:

- During site preparation and clearing,
- During excavations performed for the remedial remediation and to relocate or remove existing underground improvements at the site;
- During earthwork, including observation and acceptance of remedial removal bottoms and fill placement;
- Following the completion of grading, in order to verify soil properties for foundations, slab-on-grade and pavements;
- Upon completion of any foundation or structural excavation, prior to pouring concrete;
- During slab and flatwork subgrade preparation prior to pouring of concrete;
- During placement of backfill for utility trenches;
- During placement of backfill for retaining structures;
- During installation and backfill of subdrainage systems;
- During subgrade preparation and placement of aggregate base and asphaltic concrete; and
- When any unusual soil conditions are encountered.

3.16 Limitations

This report has been prepared for the exclusive use of our client, Huntington Beach City School District, within the scope of services requested by our client for the specific project in Huntington Beach described herein. This report or its contents should not be used or relied upon for other projects or purposes, or by other parties without the acknowledgement of NMG and the consultation of a geotechnical professional. The means and methods used by NMG for this study are based on local geotechnical standards of practice, care, and requirements of governing agencies. No warranty or guarantee, expressed or implied, is given.

Our findings, conclusions, and recommendations are professional opinions based on interpretations and inferences made from geologic and engineering data from specific locations and depths, observed or collected at a given time. By nature, geologic conditions can vary from point to point, can be very different in-between exploration points, and can also change over time. Our conclusions and recommendations are, by nature, preliminary and subject to verification and/or modification by NMG during grading and construction when more subsurface data is exposed.



Liquefaction

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides

Areas where previous occurance of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

SITE LOCATION AND SEISMIC HAZARDS MAP

BASE: U.S.G.S. SEISMIC HAZARDS MAP,
NEWPORT BEACH QUADRANGLE
Dated: April 17, 1997

N

LE BARD PARK CITY OF HUNTINGTON BEACH, CALIFORNIA	Project Number: 11117-01 Project Name: HBCSD/LE BARD PARK Date: 1-31-12 Figure No. 1	NMG Geotechnical, Inc.

LEGEND

Base: Google Maps 3-7-11

N

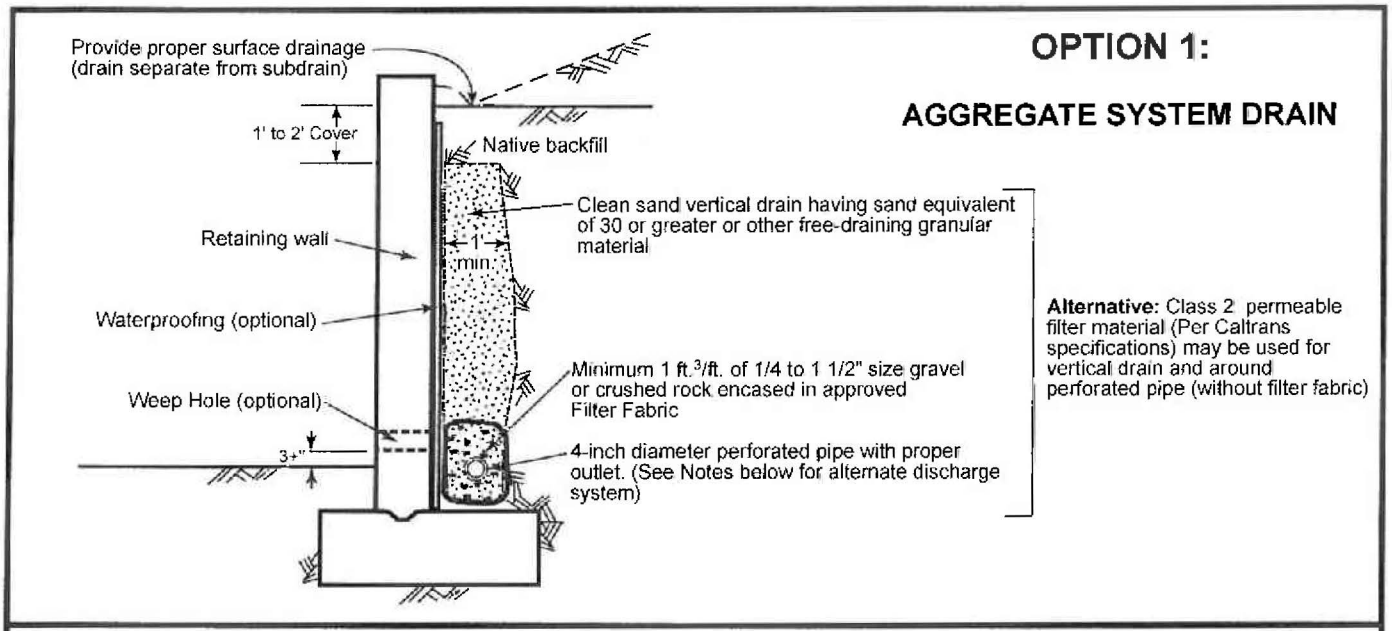
● **B-4** **T.D 51.5'** — HOLLOW STEM AUGER BORING BY NMG, SHOWING TOTAL DEPTH

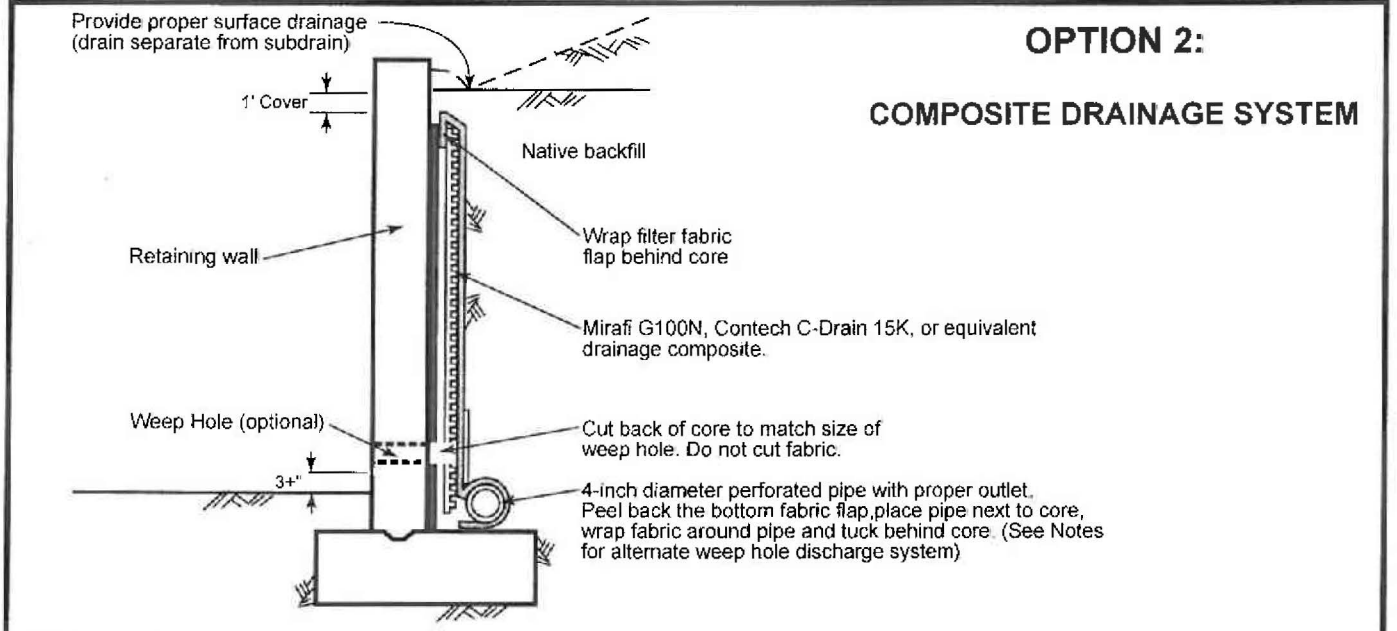
▲ **CPT-2** **T.D 50'** — CONE PENETROMETER TEST BY NMG, SHOWING TOTAL DEPTH

BORING LOCATION MAP
LE BARD PARK
CITY OF HUNTINGTON BEACH, CALIFORNIA

Project Number: 11117-01
Project Name: HBCSD/LE BARD PARK
Date: 1-31-12 Figure No. 2

NMG Geotechnical, Inc.





NOTES:

1. **PIPE TYPE SHOULD BE PVC OR ABS, SCHEDULE 40 OR SDR35 SATISFYING THE REQUIREMENTS OF ASTM TEST STANDARD D1527, D1785, D2751 , OR D3034.**
2. **FILTER FABRIC SHALL BE APPROVED PERMEABLE NON-WOVEN POLYESTER, NYLON, OR POLYPROPYLENE MATERIAL.**
3. **DRAIN PIPE SHOULD HAVE A GRADIENT OF 1 PERCENT MINIMUM.**
4. **WATERPROOFING MEMBRANE MAY BE REQUIRED FOR A SPECIFIC RETAINING WALL (SUCH AS A STUCCO OR BASEMENT WALL).**
5. **WEEP HOLES MAY BE PROVIDED FOR LOW RETAINING WALLS (LESS THAN 3 FEET IN HEIGHT) IN LIEU OF A VERTICAL DRAIN AND PIPE AND WHERE POTENTIAL WATER FROM BEHIND THE RETAINING WALL WILL NOT CREATE A NUISANCE WATER CONDITION. IF EXPOSURE IS NOT PERMITTED, A PROPER SUBDRAIN OUTLET SYSTEM SHOULD BE PROVIDED.**
6. **IF EXPOSURE IS PERMITTED, WEEP HOLES SHOULD BE 2-INCH MINIMUM DIAMETER AND PROVIDED AT 25-FOOT MAXIMUM SPACING ALONG WALL. WEEP HOLES SHOULD BE LOCATED 3+ INCHES ABOVE FINISHED GRADE.**
7. **SCREENING SUCH AS WITH A FILTER FABRIC SHOULD BE PROVIDED FOR WEEP HOLES/OPEN JOINTS TO PREVENT EARTH MATERIALS FROM ENTERING THE HOLES/JOINTS.**
8. **OPEN VERTICAL MASONRY JOINTS (I.E., OMIT MORTAR FROM JOINTS OF FIRST COURSE ABOVE FINISHED GRADE) AT 32-INCH MAXIMUM INTERVALS MAY BE SUBSTITUTED FOR WEEP HOLES.**
9. **THE GEOTECHNICAL CONSULTANT MAY PROVIDE ADDITIONAL RECOMMENDATIONS FOR RETAINING WALLS DESIGNED FOR SELECT SAND BACKFILL.**

RETAINING WALL DRAINAGE DETAIL

NMG
Geotechnical, Inc.

3/06 RETAINING WALL DRAINAGE.ai

FIGURE 3

APPENDIX A

APPENDIX A

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APPENDIX B

Report HOLLOW STEM; Project P:\2011\11117-01\GINT\11117-01 GPJ; Data Template NMGNOV98.GDT; Printed: 1/31/12

Date(s) Drilled	12/22/11	Logged By	PA
Drilling Company	2R Drilling	Drill Bit Size/Type	8"
Drill Rig Type	CME 75	Hammer Data	140 lbs @ 30" Drop
Sampling Method(s)	Modified California, Bulk		
Approximate Groundwater Depth:	34.5 Feet at End of Drilling		
Comments	Groundwater at 22.5 Feet After 30 Minutes.		

H-1

Sheet 1 of 2

Total Depth Drilled (ft)	51.5
Approximate Ground Surface Elevation (ft)	9.0

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0					ML	Surface: Grass **Alluvium (Qal)**			
			D-1	11			@ 2.5' Olive brown fine-grained sandy SILT, very moist, medium stiff, massive, slight FeO staining, few root-hairs.	20.5	102.5	B-1 @ 0'- 5'
	5		D-2	7		ML/MH	@ 5' Olive brown clayey SILT, wet to saturated, soft, FeO staining, micaceous, massive.	42.6	80.4	GS, AL
			D-3	5			@ 7.5' Olive brown SILT, wet to saturated, soft, FeO staining, micaceous, massive.	44.4	75.3	CN
0	10		D-4	6			@ 10' Olive brown SILT, wet to saturated, soft, FeO staining, micaceous, massive.	43.9	77.8	
	15		D-5	7		ML	@ 15' Olive gray fine-grained sandy SILT, wet, FeO staining, micaceous, massive.	34.9	86.4	
-10	20		D-6	6			@ 20' Dark gray SILT, wet, soft, micaceous, massive. ▽	35.2	83.9	
	25		D-7	5		CL	@ 25' Gray silty CLAY, saturated, soft, micaceous, massive, plastic, shell fragments.	37.1	84.5	
-20	30									

LOG OF BORING

TRG/Le Bard
Huntington Beach, California
PROJECT NO. 11117-01

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Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	30		D-8	7		CL-ML	@ 30' Gray silty CLAY/ clayey SILT, saturated, soft, massive, micaceous, shell fragments.	34.5	90.8	
	35		D-9	11		ML	@ 35' Gray fine-grained sandy SILT, saturated, micaceous, massive.	27.4	94.3	GS
-30										
	40		D-10	7		CL	@ 40' Dark gray silty CLAY, saturated, soft, micaceous, shell fragments.	37.5	84.4	
	45		D-11	16		ML	@ 45' Dark gray fine-grained sandy SILT, saturated, medium stiff, massive, micaceous, few organics.	25.3	100.2	
-40										
	50		D-12	12		SM	@ 50' Upper: Dark gray fine-grained sandy SILT, saturated, medium stiff, massive, micaceous, few organics.	33.7	89.1	
							Lower: Gray fine-grained silty SAND, saturated, medium dense, massive.			
	55									
-50										
	60									
	65									

Notes:
Total Depth: 51.5 Feet.
Groundwater At 22.5 Feet.
Backfilled with Cuttings.

LOG OF BORING

TRG/Le Bard
Huntington Beach, California

PROJECT NO. 11117-01

NMG

Report: HOLLOW STEM; Project: P:\2011\11117-01\GINT\11117-01.GPJ; Data Template: NMGNOV98.GDT; Printed: 1/31/12

Date(s) Drilled	12/22/11	Logged By	PA
Drilling Company	2R Drilling	Drill Bit Size/Type	8"
Drill Rig Type	CME 75	Hammer Data	140 lbs @ 30" Drop
Sampling Method(s)	Modified California, Bulk		
Approximate Groundwater Depth:	Not Available		
Comments	Water Added to Boring		

H-2

Sheet 1 of 2

Total Depth Drilled (ft)	31.5
Approximate Ground Surface Elevation (ft)	9.0

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0					ML	Surface: Turf **Alluvium (Qal)**			
			D-1	10			@ 2.5' Brown fine-grained sandy SILT, moist, medium stiff, micaceous, massive, roots.	23.0	92.3	B-1 @ 0'- 5' EI, MD, GS, CC
	5		D-2	8		CL	@ 5' Gray CLAY, wet to saturated, medium stiff, slight FeO staining, massive, micaceous.	38.4	86.1	DS
			D-3	4		ML	@ 7.5' Olive gray clayey SILT, saturated, soft, micaceous, massive, root-hairs.	35.9	80.6	
0	10		D-4	6			@ 10' Olive gray SILT, saturated, soft, micaceous, massive, root-hairs.	48.6	75.2	
	15		D-5	3		CL	@ 15' Gray silty CLAY, saturated, soft, MnO, massive, highly plastic.	52.6	69.1	
-10	20		D-6	6		ML	@ 20' Dark gray fine-grained sandy SILT, saturated, soft, massive, highly micaceous.	36.5	85.6	
	25		D-7	7			@ 25' Dark gray clayey SILT, saturated, soft, massive, plastic, shell fragments.	35.5	87.3	
-20	30									

LOG OF BORING

TRG/Le Bard
Huntington Beach, California
PROJECT NO. 11117-01

NMG

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	30		D-8	11		ML	@ 30' Dark gray clayey SILT, saturated, soft, massive, plastic, shell fragments	35.5	91.4	
	35						Notes: Total Depth: 31.5 Feet. Groundwater Not Available (Broken Irrigation Line). Backfilled with Cuttings.			
-30	40									
	45									
-40	50									
	55									
-50	60									
	65									

LOG OF BORING

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PROJECT NO. 11117-01

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Report: HOLLOW STEM; Project: P:\2011\11117-01\GINT\11117-01.GPJ; Data Template: NMGNOV98.GDT; Printed: 1/31/12

Date(s) Drilled	12/22/11	Logged By	PA
Drilling Company	2R Drilling	Drill Bit Size/Type	8"
Drill Rig Type	CME 75	Hammer Data	140 lbs @ 30" Drop
Sampling Method(s)	Modified California, Bulk		
Approximate Groundwater Depth:	Not Available		
Comments	Boring left open 8 Hours (Caved to 5 Feet).		

H-3

Sheet 1 of 2

Total Depth Drilled (ft)	51.5
Approximate Ground Surface Elevation (ft)	9.0

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0					ML/MH	Surface: Asphaltic Concrete and Aggregate Base **Alluvium (Qal)**			
			D-1	12			@ 2.5' Olive gray SILT, moist, medium stiff, micaceous, massive, slight FeO staining.	28.8	90.6	DS, CN B-1 @ 0'- 5' (MD, GS, AL, EI, RV)
	5		D-2	9		ML	@ 5' Olive gray fine-grained sandy SILT, moist, medium stiff, massive, micaceous.	32.0	86.2	
0			D-3	6		CL	@ 7.5' Gray CLAY, wet to saturated, soft, highly plastic, micaceous, massive.	44.4	78.3	
	10		D-4	9		ML	@ 10' Gray fine-grained sandy SILT, wet, medium stiff, micaceous, massive.	34.7	87.5	
	15		D-5	5		CL/ML	@ 15' Gray silty CLAY/ clayey SILT, saturated, soft, massive, plastic.	41.5	78.1	
-10										
	20		D-6	12		ML	@ 20' Dark gray SILT, saturated, medium stiff, micaceous, massive.	28.6	95.7	
	25		D-7	7			@ 25' Dark gray SILT, saturated, medium stiff, micaceous, massive, shell fragments.	31.5	90.6	
-20	30									

LOG OF BORING

TRG/Le Bard
Huntington Beach, California
PROJECT NO. 11117-01

NMG

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	30		D-8	11			@ 30' Dark gray SILT, saturated, medium stiff, micaceous, massive.	34.7	88.9	
	35		D-9	15			@ 35' Gray SILT, saturated, medium stiff, massive, micaceous.	28.2	95.6	
-30	40		D-10	8		CL	@ 40' Gray silty CLAY, saturated, soft, massive, abundant shell fragments, micaceous, plastic.	39.1	80.7	
	45		D-11	19		ML	@ 45' Gray SILT, saturated, medium stiff, micaceous, massive.	33.5	88.2	
-40	50		D-12	14			@ 50' Gray fine-grained sandy SILT, saturated, medium dense, locally sand, trace organics, massive.	27.5	97.5	
							Notes: Total Depth: 51.5 Feet. Groundwater Not Encountered (Hole Caved to 5 Feet). Backfilled with Cuttings.			
	55									
-50	60									
	65									

LOG OF BORING

TRG/Le Bard
Huntington Beach, California

PROJECT NO. 11117-01

NMG

Report: HOLLOW STEM; Project: P:\2011\11117-01\GINT\11117-01.GPJ; Data Template: NMGNOV98.GDT; Printed: 1/31/12

Date(s) Drilled	12/22/11	Logged By	PA
Drilling Company	2R Drilling	Drill Bit Size/Type	8"
Drill Rig Type	CME 75	Hammer Data	140 lbs @ 30" Drop
Sampling Method(s)	Modified California, Bulk		
Approximate Groundwater Depth:	38.3 Feet at end of Drilling		
Comments			

H-4

Sheet 1 of 2

Total Depth Drilled (ft)	51.5
Approximate Ground Surface Elevation (ft)	9.0

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	0					ML	Surface: Grass **Undocumented Artificial Fill (Afu)**			
			D-1	11			@ 2.5' Brown sandy SILT, moist, medium dense, abundant roots up to 0.5" in diameter.	20.5	96.5	B-1 @ 0'- 5' (GS)
	5		D-2	6		ML	**Alluvium (Qal)** @ 5' Olive fine-grained sandy SILT, moist, soft, micaceous, massive, FeO staining.	29.8	86.7	
0			D-3	4		CL	@ 7.5' Brownish gray CLAY, saturated, soft, massive, slight FeO staining.	51.1	72.4	
	10		D-4	8			@ 10' Very dark gray CLAY, saturated, soft, medium stiff, slight FeO staining, micaceous, massive.	44.3	80.0	
	15		D-5	2		CH	@ 15' Gray highly plastic CLAY, saturated, soft, massive, few organics, micaceous.	47.8	74.1	
-10	20		D-6	11		ML	@ 20' Gray fine-grained sandy SILT, saturated, medium stiff, highly micaceous, massive, shell fragments.	28.8	91.5	
	25		D-7	8		CL	@ 25' Dark gray silty CLAY, saturated, soft to medium stiff, highly micaceous, massive, shell fragments.	38.2	83.6	
-20	30									

LOG OF BORING

TRG/Le Bard
Huntington Beach, California
PROJECT NO. 11117-01

NMG

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	OTHER TESTS and REMARKS
	30		D-8	13		ML	@ 30' Gray clayey SILT, saturated, medium stiff, micaceous, massive.	28.2	97.9	
	35		D-9	10			@ 35' Dark gray SILT, saturated, medium stiff, highly micaceous, massive. ∇	32.7	90.7	
-30	40		D-10	8		CL	@ 40' Dark gray CLAY, saturated, soft to medium stiff, massive, micaceous, shell fragments.	37.7	83.3	
	45		D-11	25		SM-ML	@ 45' Gray fine-grained silty SAND/ sandy SILT, saturated, medium dense to stiff, massive, micaceous.	29.3	93.6	GS
-40	50		D-12	29		ML	@ 50' Gray SILT, saturated, stiff, massive, highly micaceous.	31.1	90.5	
	55						Notes: Total Depth: 51.5 Feet. Groundwater At 38.3 Feet. Backfilled with Cuttings.			
-50	60									
	65									

LOG OF BORING

TRG/Le Bard
Huntington Beach, California

PROJECT NO. 11117-01

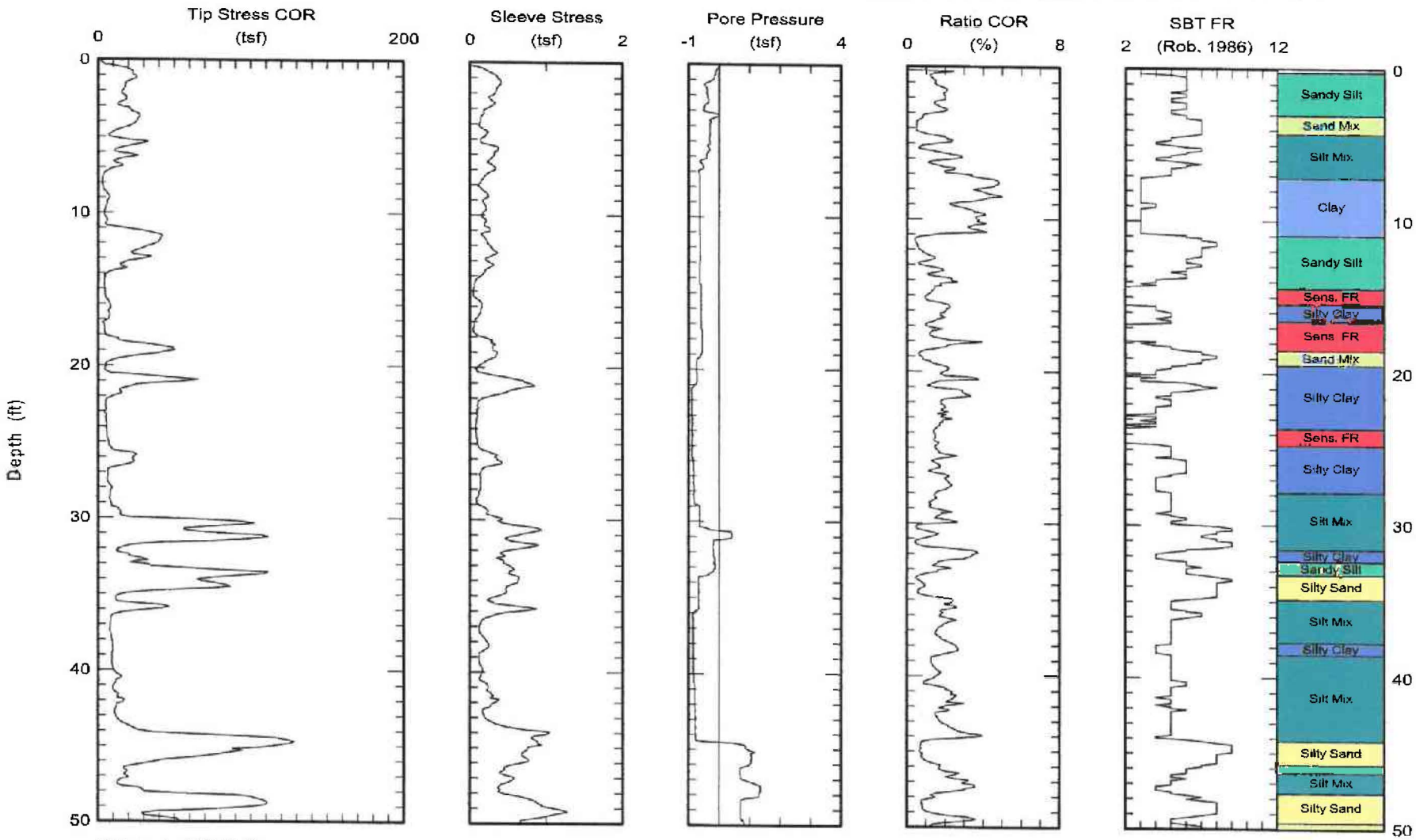
NMG

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
www.kehoetesting.com

CPT Data
30 ton rig

Date: 22/Dec/2011
Test ID: CPT-1
Project: HuntingtonBeach

Customer: NMG Geotechnical, Inc.
Job Site: Le Bard Park.



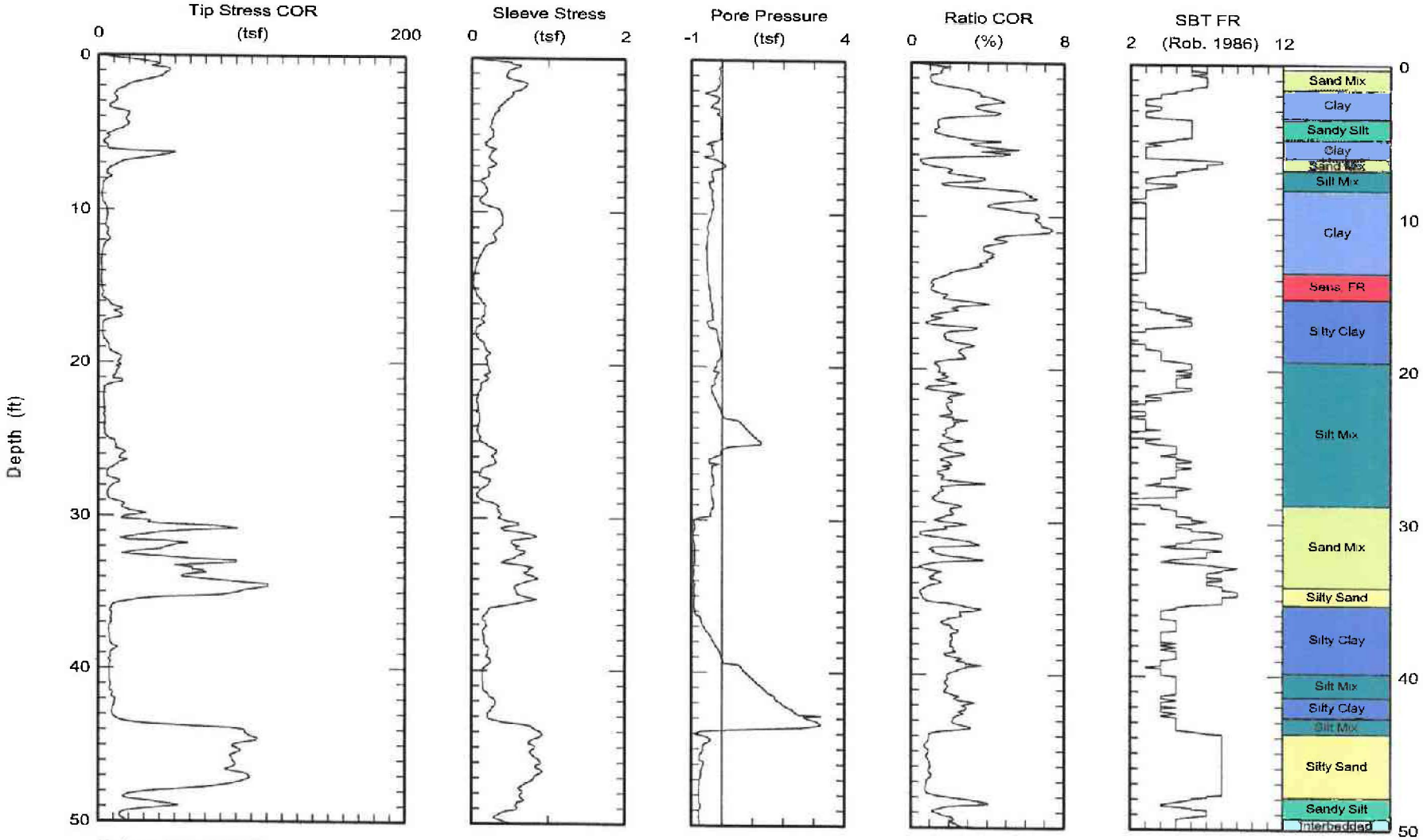
Maximum depth: 50.29 (ft)

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
www.kehoetesting.com

CPT Data
30 ton rig

Date: 22/Dec/2011
Test ID: CPT-2
Project: HuntingtonBeach

Customer: NMG Geotechnical, Inc.
Job Site: Le Bard Park.



Maximum depth: 50.40 (ft)

Test ID: CI

APPENDIX C

Sample	Compacted Moisture (%)	Compacted Dry Density (pcf)	Final Moisture (%)	Volumetric Swell (%)	Expansion Index[1] Value	Method	Expansive Classification[2]	Soluble Sulfate (%)	Sulfate Exposure[3]
H-2 B-1 0-5'	12.5	103.6	23.0	3.2	34	B	Low	0.062	Negligible
H-3 B-1 0-5'	13.0	105.7	21.6	1.9	24	B	Low	0.05	Negligible

Test Method:

ASTM D4829

HACH SF-1 (Turbidimetric)

Notes:

1. Expansion Index (EI) method of determination:
   [A] E.I. determined by adjusting water content to achieve a 50 ±1% degree of saturation
   [B] E.I. calculated based on measured saturation within the range of 40% and 60%
2. ASTM D4829 *(Classification of Expansive Soil)*
3. ACI-318 Table 4.3.1 *(Requirement for Concrete Exposed to Sulfate-Containing Solutions)*

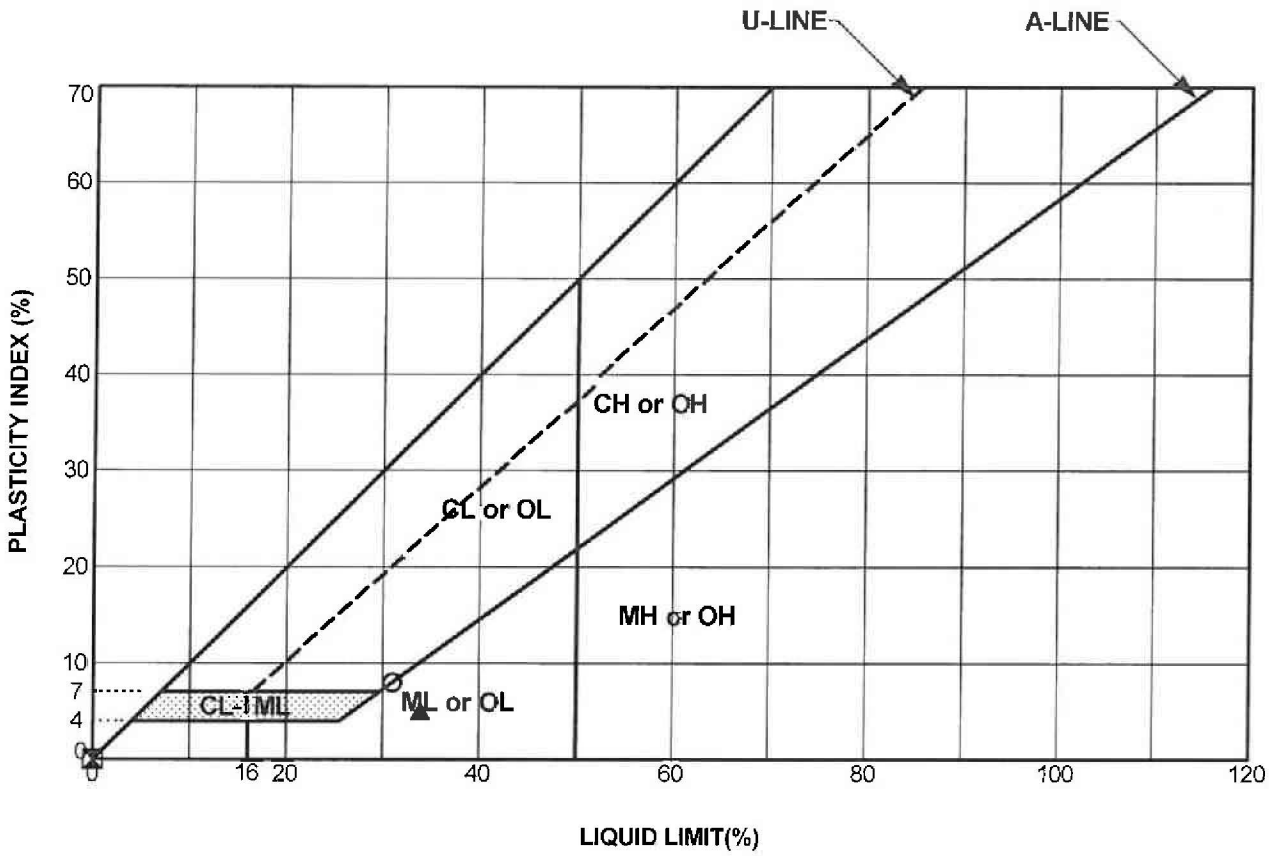
Expansion Index and Soluble Sulfate Test Results

(FRM001 Rev.5)

Project No. TRG / Le Bard

Project Name: 11117-01

NMG

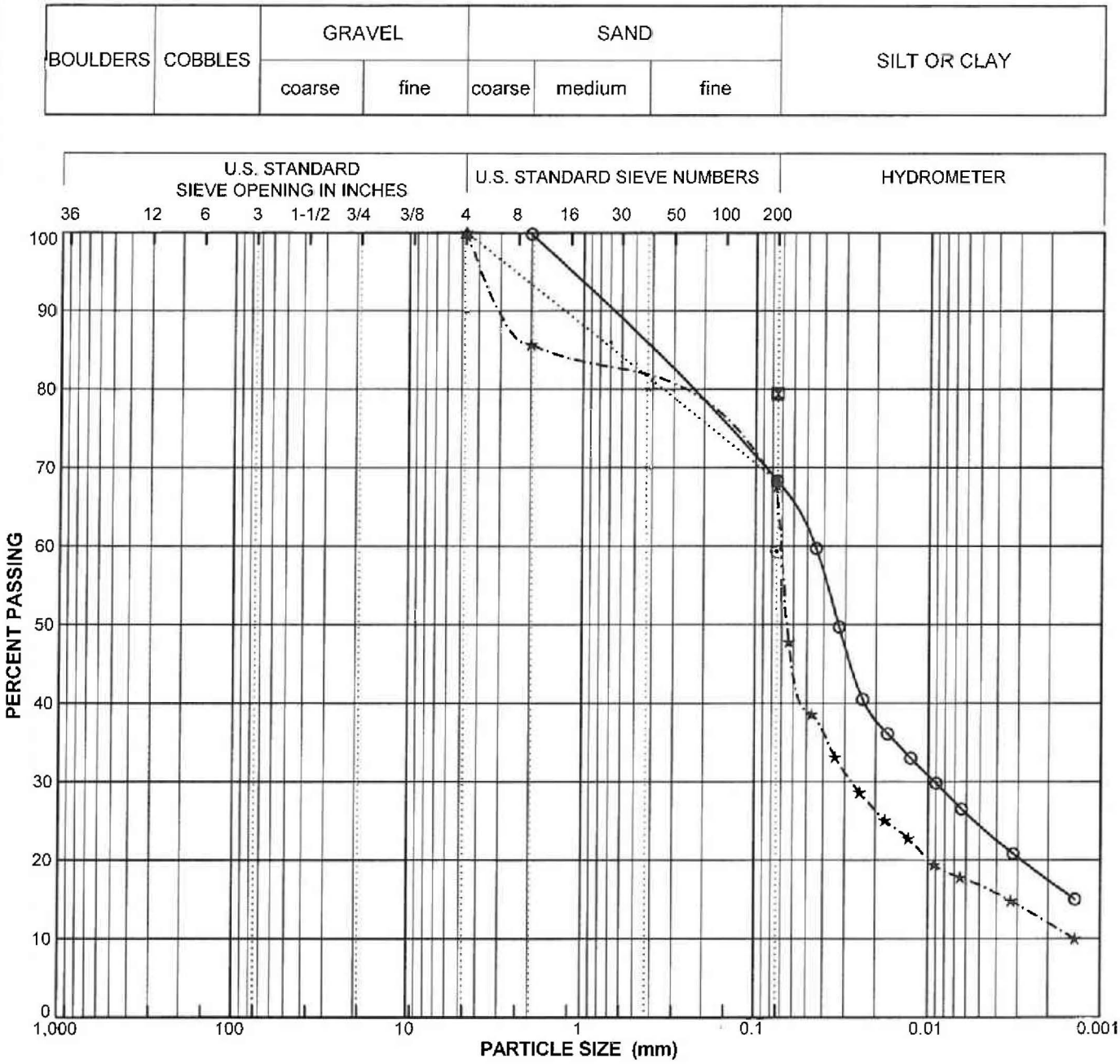


Symbol	Boring Number	Depth (feet)	Sample Number	Passing No. 200 Sieve (%)	LL	PI	USCS	Description
○	H-1	5.0	D-2	68	31	8	ML/MH	(Qal) Olive Brown Clayey SILT
⊠	H-1	35.0	D-9	79	NP	NP	ML	(Qal) Gray Sandy SILT
▲	H-3	2.0	B-1	68	34	5	MH	(Qal) Olive Gray SILT

PLASTICITY CHART

TRG/Le Bard
Huntington Beach, California
PROJECT NO. 11117-01

NMG Geotechnical, Inc.



Symbol	Boring Number	Sample Number	Depth (feet)	Field Moisture (%)	LL	PI	Activity PI/-2μ	C_u	C_c	Passing No. 200 Sieve (%)	Passing 2μ (%)	USCS
○	H-1	D-2	5.0	43	31	8				68	18	ML/MH
⊠	H-1	D-9	35.0	27	NP	NP				79		ML
▲	H-2	B-1	2.0							68		ML
★	H-3	B-1	2.0		34	5				68	12	MH
⊙	H-4	D-1	2.5	21						59		ML

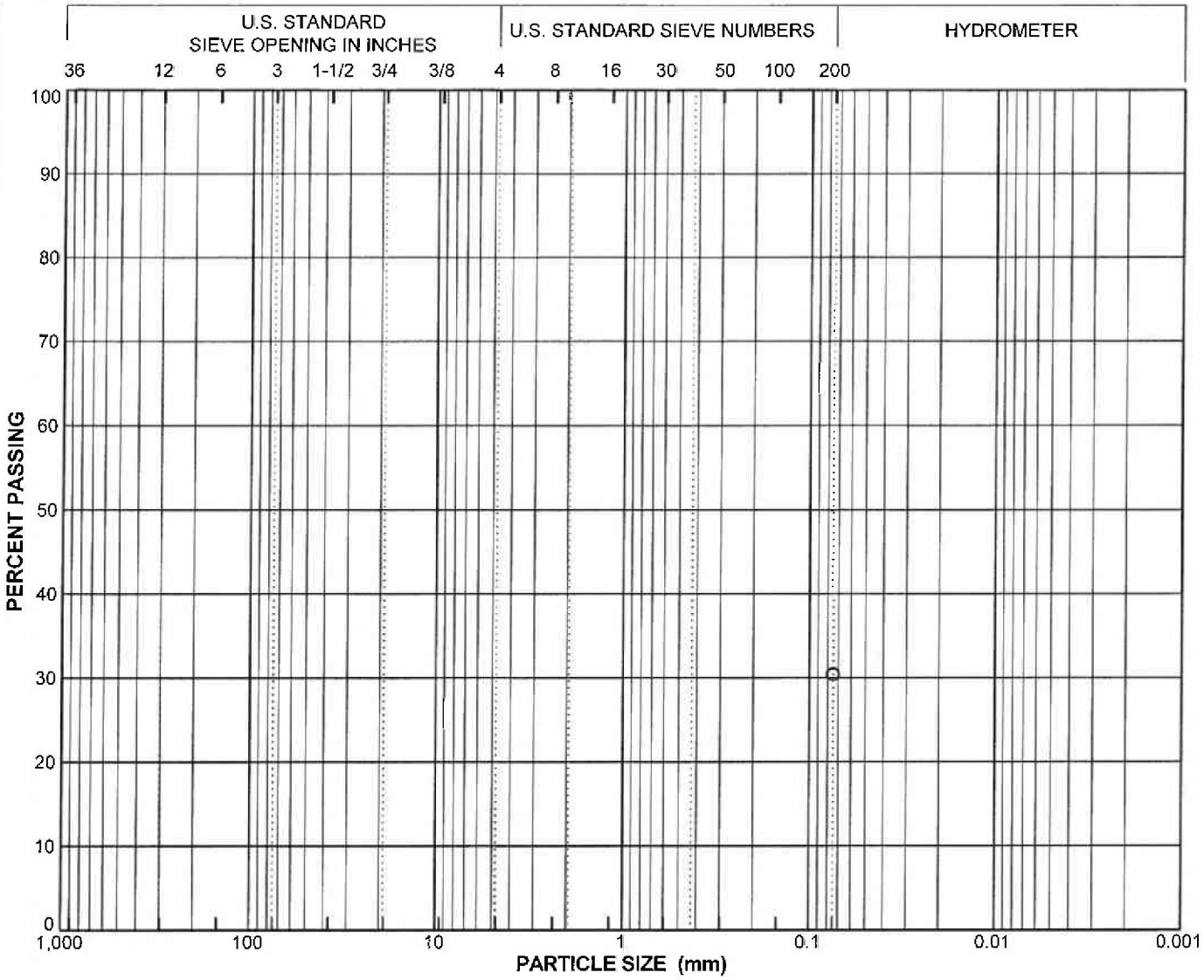
PARTICLE SIZE DISTRIBUTION

TRG/Le Bard
Huntington Beach, California
PROJECT NO. 11117-01

NMG Geotechnical, Inc.

Template: NMSIV; Prj ID: 11117-01.GPJ; Printed: 1/20/12

BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		coarse	fine	coarse	medium	fine	



Symbol	Boring Number	Sample Number	Depth (feet)	Field Moisture (%)	LL	PI	Activity PI/-2μ	C_u	C_c	Passing No. 200 Sieve (%)	Passing 2μ (%)	USCS
O	H-4	D-11	45.0	29						30		SM

PARTICLE SIZE DISTRIBUTION

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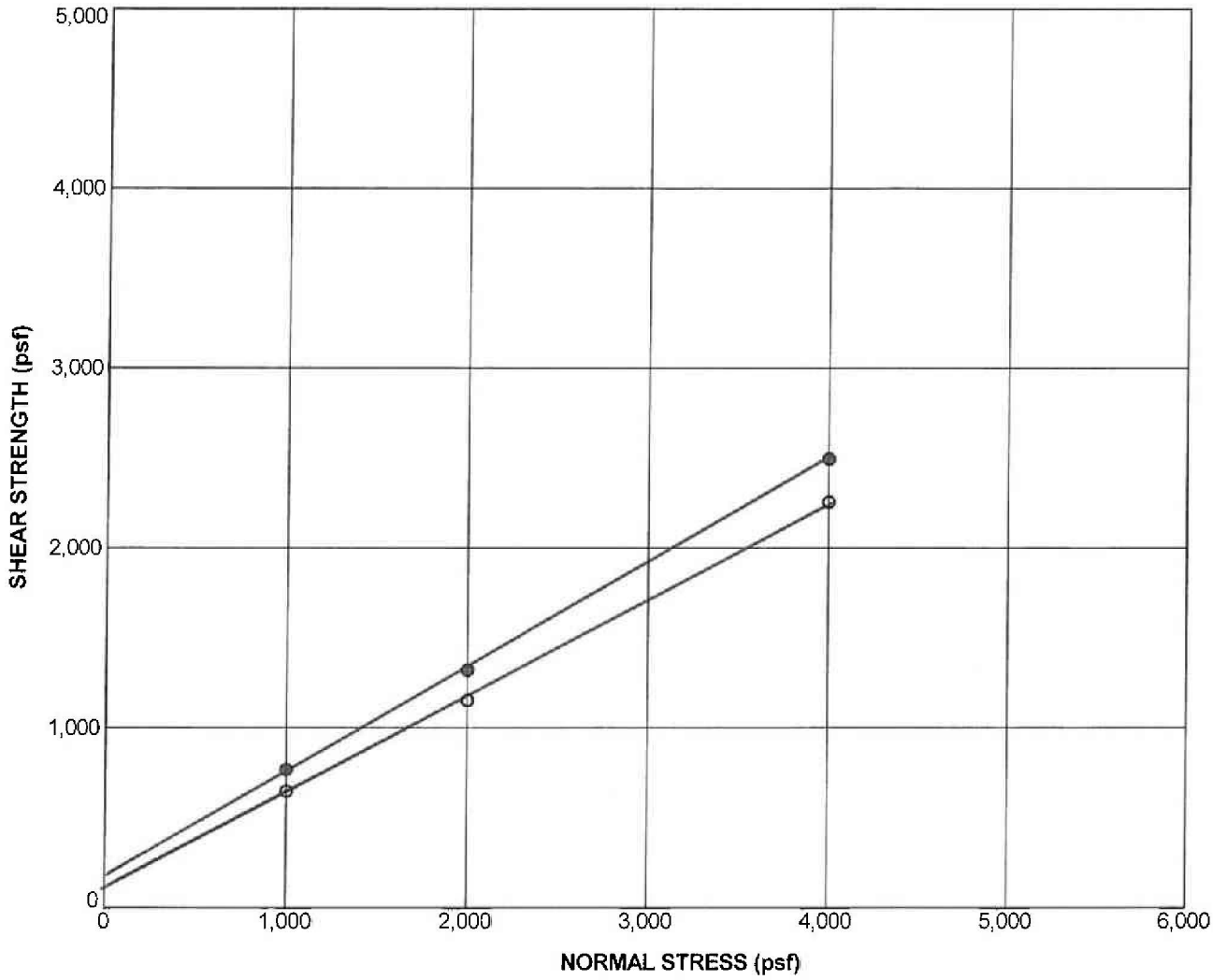
Huntington Beach, California
PROJECT NO. 11117-01

NMG Geotechnical, Inc.



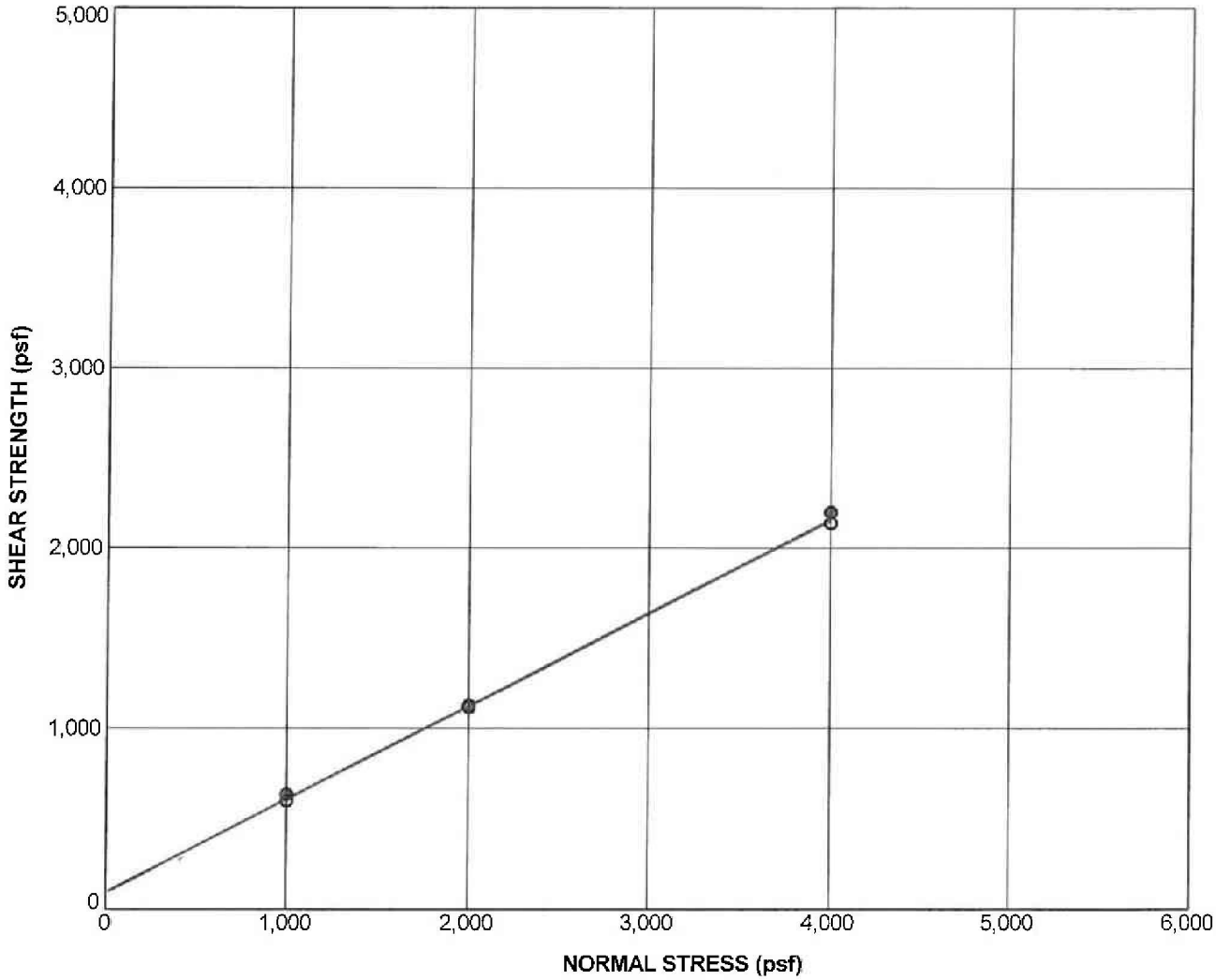
Boring No. H-2		Sample No. D-2		Depth: 5.0 ft	
Sample Description:	(Qal) Olive Brown Clayey SILT				
Liquid Limit:		Plasticity Index:		Percent Passing No. 200 Sieve:	
Moisture Content (%):	36.4	Dry Density (pcf):	87.3	Degree of Saturation (%):	100
Sample Type:	Undisturbed	Rate of Shear (in./min.):	0.005		

SHEAR STRENGTH PARAMETERS		
Parameter	Peak ●	Ultimate ○
Cohesion (psf)	130	100
Friction Angle (degrees)	31	29.0

DIRECT SHEAR TEST RESULTS

TRG/Le Bard
Huntington Beach, California
PROJECT NO. 11117-01

NMG Geotechnical, Inc.



Boring No. H-3		Sample No. D-1		Depth: 2.5 ft	
Sample Description:	(Qal) Dark Olive Clayey SILT				
Liquid Limit:		Plasticity Index:		Percent Passing No. 200 Sieve:	
Moisture Content (%):	37.9	Dry Density (pcf):	84.3	Degree of Saturation (%):	99
Sample Type:	Undisturbed	Rate of Shear (in./min.):	0.005		

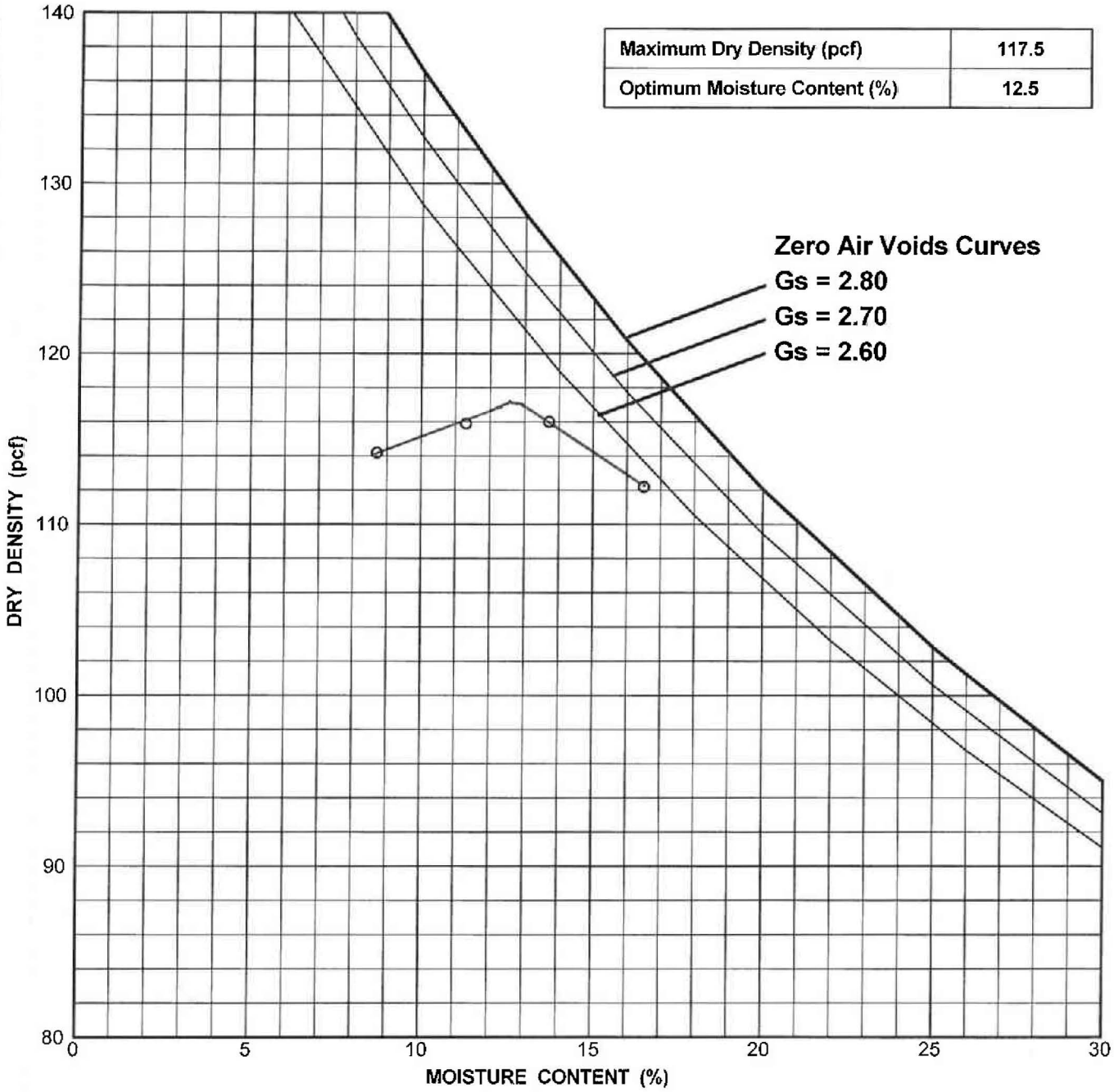
SHEAR STRENGTH PARAMETERS

Parameter	Peak ●	Ultimate ○
Cohesion (psf)	80	80
Friction Angle (degrees)	28	28.0

DIRECT SHEAR TEST RESULTS

TRG/Le Bard
Huntington Beach, California
PROJECT NO. 11117-01

NMG Geotechnical, Inc.

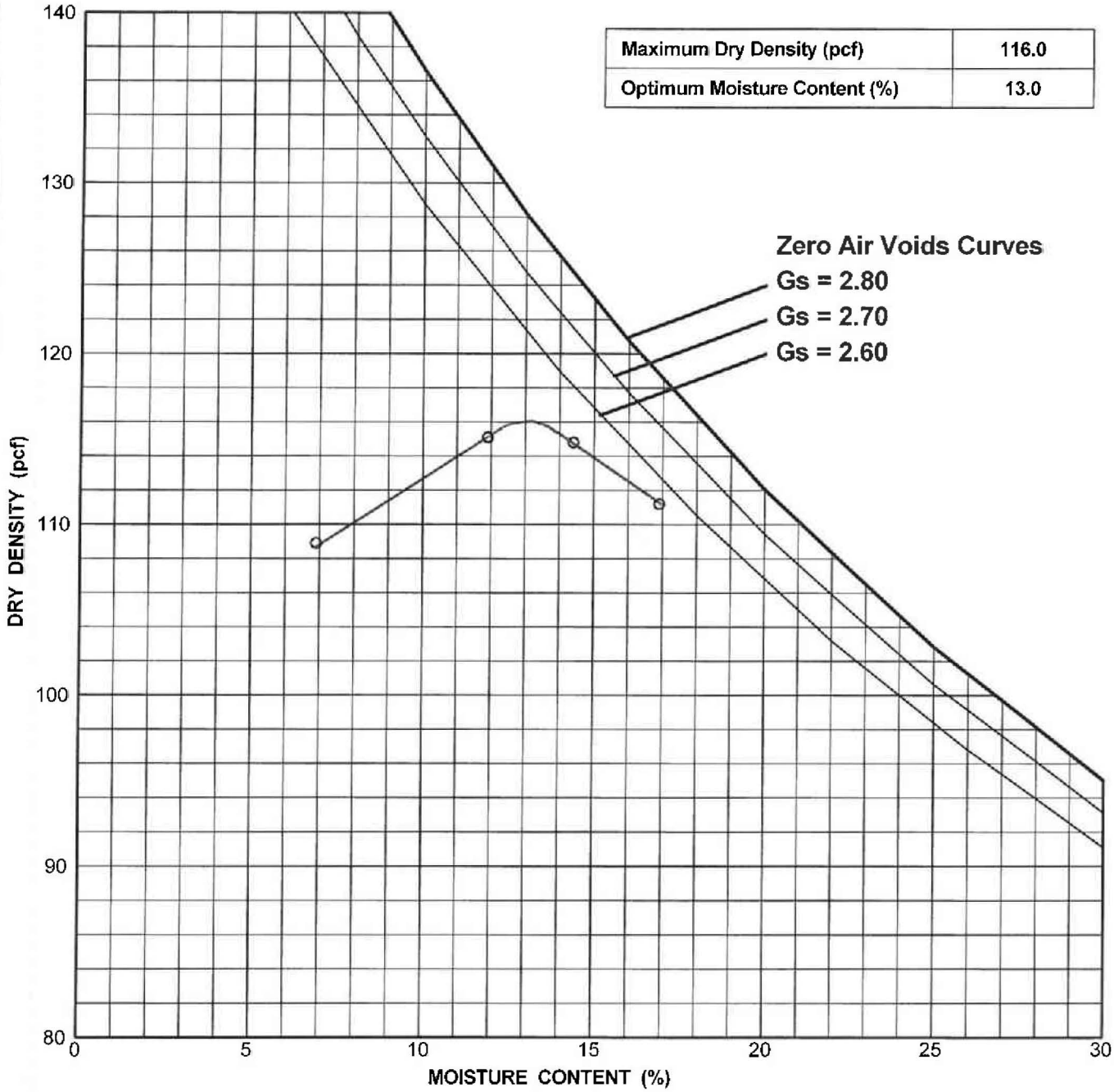


Boring No. H-2	**Sample No. B-1**	**Depth: 2.0 ft**
Sample Description: (Qal) Pale Brown Sandy SILT		
Liquid Limit:	**Plasticity Index:**	**Percent Passing No. 200 Sieve:** 68
Comments: 1557A		

COMPACTION TEST RESULTS

TRG/Le Bard
Huntington Beach, California
PROJECT NO. 11117-01

NMG Geotechnical, Inc.



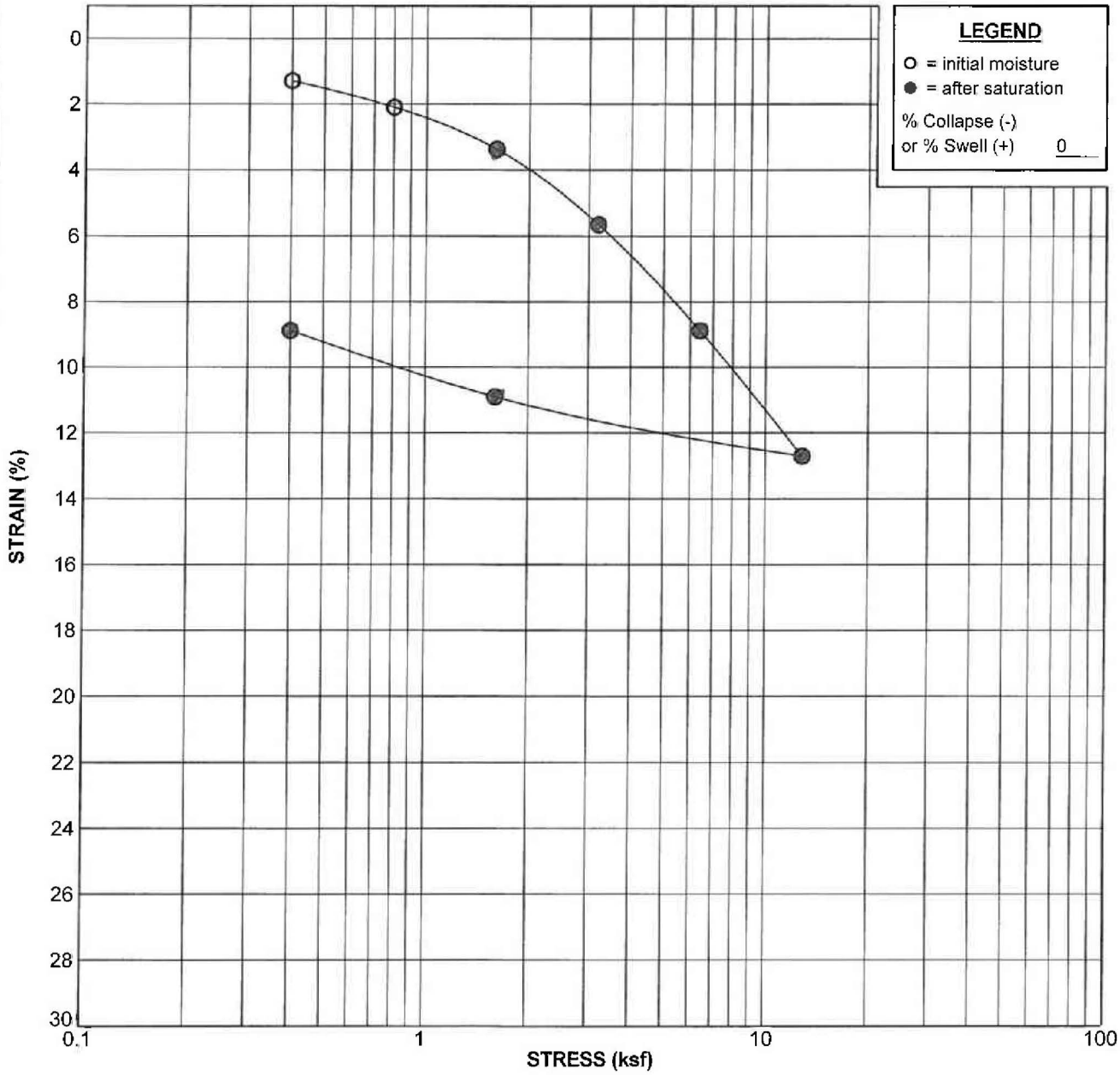
Maximum Dry Density (pcf)	116.0
Optimum Moisture Content (%)	13.0

Boring No. H-3	**Sample No. B-1**	**Depth: 2.0 ft**
Sample Description: (Qal) Olive Gray SILT		
Liquid Limit: 34	**Plasticity Index:** 5	**Percent Passing No. 200 Sieve:** 68
Comments: 1557A		

COMPACTION TEST RESULTS

TRG/Le Bard
Huntington Beach, California
PROJECT NO. 11117-01

NMG Geotechnical, Inc.



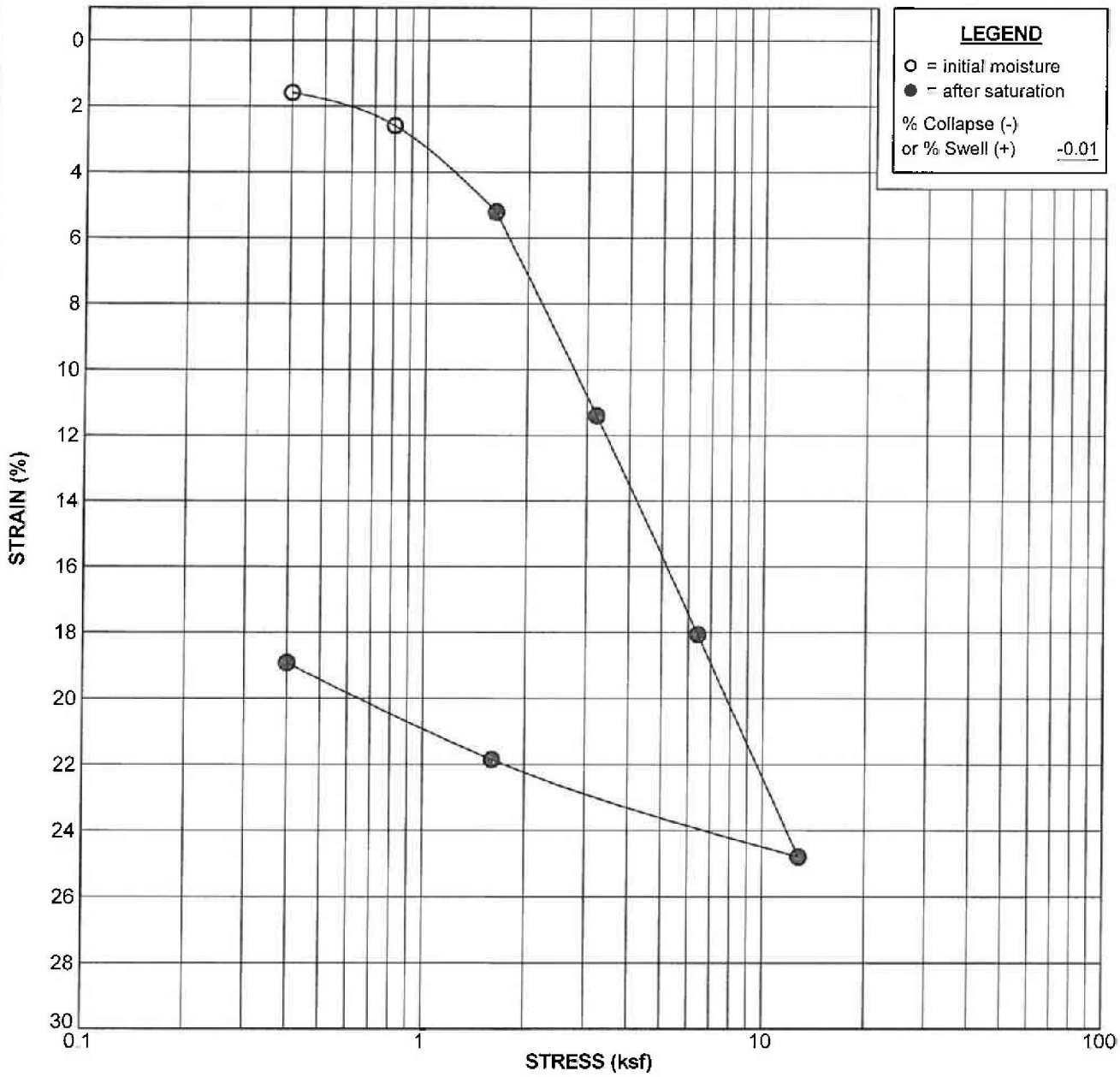
Boring No. H-1	Sample No. D-3	Depth: 7.5 ft
Sample Description:	(Qal) Olive Brown SILT	
Liquid Limit:	Plasticity Index:	Percent Passing No. 200 Sieve:

Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	43.2	77.5	101.0	1.134
Final	38.6	84.4	106.6	0.959

CONSOLIDATION TEST RESULTS

TRG/Le Bard
Huntington Beach, California
PROJECT NO. 11117-01

NMG Geotechnical, Inc.



Boring No. H-2	Sample No. D-5	Depth: 15.0 ft
Sample Description:	(Qal) Gray Silty CLAY	
Liquid Limit:	Plasticity Index:	Percent Passing No. 200 Sieve:

Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	55.8	66.1	98.5	1.502
Final	40.0	78.6	96.0	1.104

CONSOLIDATION TEST RESULTS

TRG/Le Bard
Huntington Beach, California
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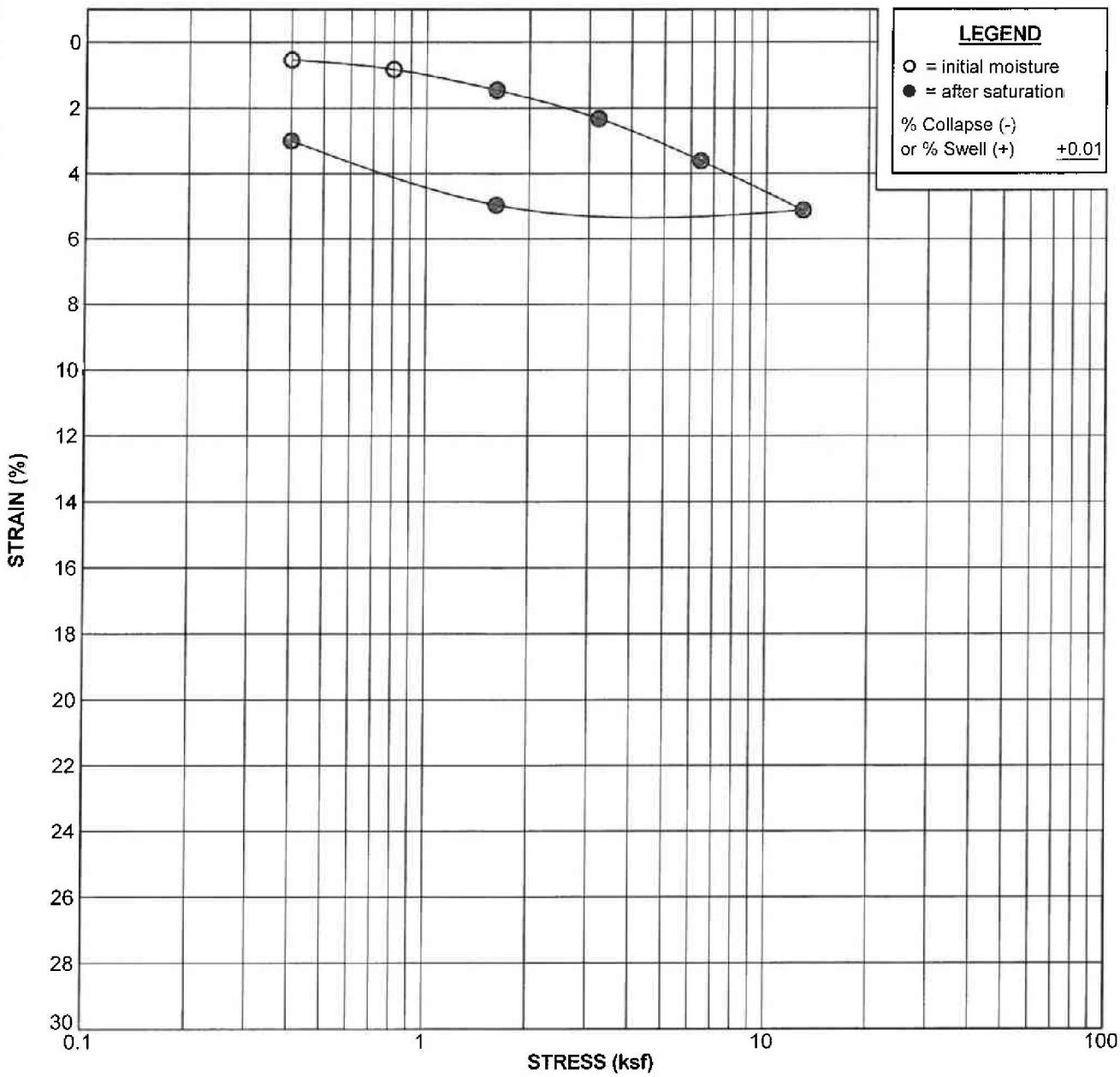
NMG Geotechnical, Inc.



Boring No. H-3	Sample No. D-1	Depth: 2.5 ft
Sample Description:	(Qal) Dark Olive Clayey SILT	
Liquid Limit:	Plasticity Index:	Percent Passing No. 200 Sieve:

Test Stage	Moisture Content (%)	Dry Density (pcf)	Degree of Saturation (%)	Void Ratio
Initial	30.3	90.9	92.0	0.922
Final	31.6	93.6	102.1	0.867

CONSOLIDATION TEST RESULTS

TRG/Le Bard
Huntington Beach, California
PROJECT NO. 11117-01

NMG Geotechnical, Inc.

CHEMICAL TEST RESULTS

WORK ORDER	SAMPLE NO	DEPTH	Ph	SOLUBLE SULFATES (ppm)	SOLUBLE CHLORIDES (ppm)	MINIMUM RESISTIVITY ohm-cm
11117-01	B-1	0-5'	8.4	49.5	620	1,200

GMU GEOTECHNICAL, INC.

NMG

L-120102
11-Jan-12

R-VALUE TEST DATA CTM 301

Project: TRG/LeBard	Project No: 11117-01	Date: 1/12/2012
Boring Trench No: B-3	Sample No: B-1	Sample Depth: 0-5'
Field Description: CL/ML		
Lab Description: Olive Brown Silty CLAY (CL)		

Specimen Number	1	2	3	4
Mold Number	1	2	3	
Water Adjustment (g)	+75	+85	+95	
Compactor Pressure (psi)	275	260	190	
Exudation Pressure (psi)	429	300	240	
Gross Weight (g)	3190.9	3197.1	3177	
Mold Tare (g)	2116.6	2128.6	2113.7	
Wet Weight (g)	1074.3	1068.5	1063.3	0
Sample Height (in)	2.5	2.5	2.5	
Initial Dial Reading	0.0619	0.0991	0.0425	
Final Dial Reading	0.0691	0.1043	0.0449	
Expansion (in $x10^{-4}$)	72	52	24	0
Stability(psi) at 2,000 lbs (160 psi)	24 46	28 54	46 102	
Turns Displacement	3.94	4.31	4.45	
R-Value Uncorrected	61	53	24	#DIV/0!
R-Value Corrected	61	53	24	
Moisture Content (%)	14.0	14.7	15.6	#DIV/0!
Dry Density (pcf)	114.3	112.9	111.4	#DIV/0!
Assumed Traffic Index	4.0	4.0	4.0	4.0
G.E. by Stability	0.40	0.48	0.78	1.02
G.E. by Expansion	2.40	1.73	0.80	0.00
Gf	1.25			

Moisture Content				
Dish No.	QQ	TT	G	
Weight of Moist Soil and Dish (g)	307.7	321.5	293	
Weight of Dry Soil and Dish (g)	276.1	286.7	260.2	
Water Loss (g)	31.6	34.8	32.8	0
Weight of Dish (g)	49.6	50.5	50.4	
Dry Soil (g)	226.5	236.2	209.8	0
Moisture Content (%)	14.0	14.7	15.6	#DIV/0!

R-Value by Exudation = 53

R-Value by Expansion = 24

R-Value at Equilibrium = 24 by Expansion

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301

Remarks:

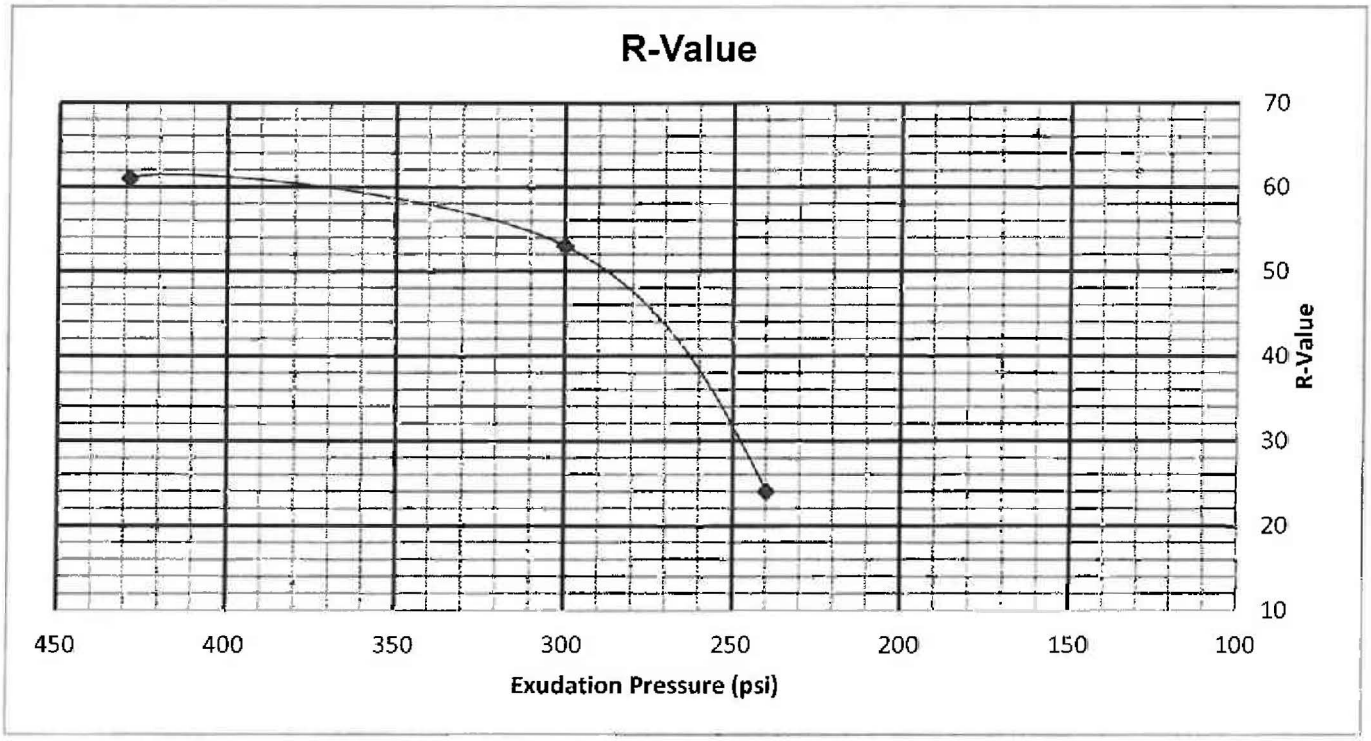
Set up by: Run by: GEH

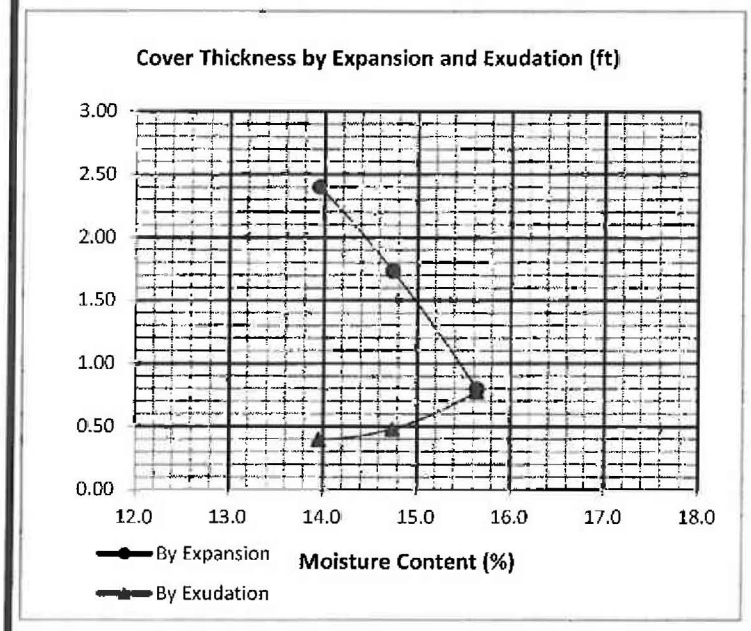
Calculated by: Checked by: Date Completed: 1/12/2012

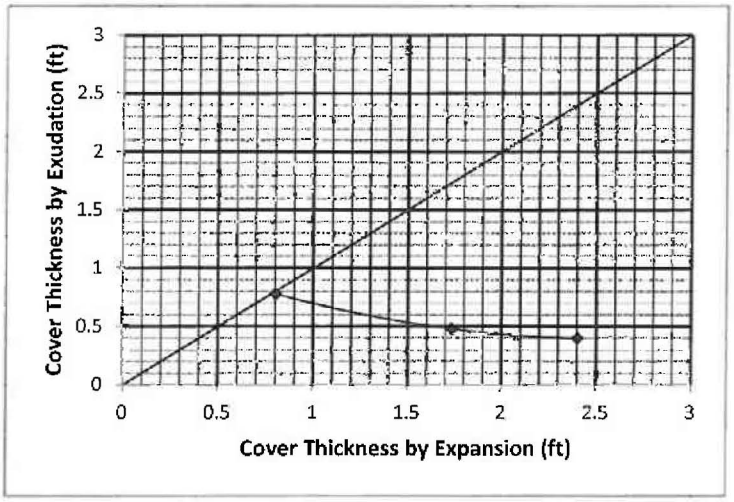
NMG Geotechnical, Inc.

R-VALUE GRAPHICAL PRESENTATION

Project: TRG/LeBard	Project No: 11117-01	Date: 1/12/2012
Boring Trench No: B-3	Sample No: B-1	Sample Depth: 0-5'
Field Description: CL/ML		
Lab Description: Olive Brown Silty CLAY (CL)		







Cover Thickness (ft) = 0.8

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301

Remarks:

Set up by: Run by: GEH

Calculated by: Checked by: Date Completed: 1/12/2012

NMG Geotechnical, Inc.

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WORK ORDER	SAMPLE NO	DEPTH	Ph	SOLUBLE SULFATES (ppm)	SOLUBLE CHLORIDES (ppm)	MINIMUM RESISTIVITY ohm-cm
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GMU GEOTECHNICAL, INC.

NMG

L-120102

11-Jan-12

APPENDIX D

USGS 2011, Seismic Design Parameters – Le Bard Park, Huntington Beach, California

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 33.6652
Longitude = -117.9511
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.01 deg grid spacing

Period (sec)	Sa (g)
0.2	1.737 (Ss, Site Class B)
1.0	0.637 (S1, Site Class B)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 33.6652
Longitude = -117.9511
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	1.737 (SMs, Site Class D)
1.0	0.956 (SM1, Site Class D)

Conterminous 48 States
2005 ASCE 7 Standard
Latitude = 33.6652
Longitude = -117.9511
Design Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	1.158 (SDs, Site Class D)
1.0	0.637 (SD1, Site Class D)

*** Deaggregation of Seismic Hazard at One Period of Spectral Accel. ***
*** Data from U.S.G.S. National Seismic Hazards Mapping Project, 2008 version ***
PSHA Deaggregation. %contributions. site: Le_Bard_Park long: 117.951 W., lat: 33.665 N.
Vs30(m/s)= 760.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.6496 g. Weight * Computed_Rate_Ex 0.412E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00796
#This deaggregation corresponds to Mean Hazard w/all GMPEs

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
8.3	5.05	0.584	0.543	0.041	0.000	0.000	0.000	0.000
8.3	5.20	1.370	1.133	0.237	0.000	0.000	0.000	0.000
8.3	5.40	1.624	1.162	0.462	0.000	0.000	0.000	0.000
8.4	5.60	1.720	1.022	0.698	0.000	0.000	0.000	0.000
15.9	5.61	0.068	0.068	0.000	0.000	0.000	0.000	0.000
8.4	5.80	1.642	0.807	0.835	0.000	0.000	0.000	0.000
16.0	5.80	0.104	0.104	0.000	0.000	0.000	0.000	0.000
7.3	6.01	2.065	0.666	1.354	0.045	0.000	0.000	0.000
14.1	6.03	0.322	0.315	0.007	0.000	0.000	0.000	0.000
6.6	6.20	2.459	0.497	1.800	0.162	0.000	0.000	0.000
12.9	6.20	0.783	0.664	0.119	0.000	0.000	0.000	0.000
6.9	6.40	2.496	0.423	1.731	0.342	0.000	0.000	0.000
12.8	6.39	0.931	0.639	0.292	0.000	0.000	0.000	0.000
4.5	6.61	12.595	1.355	5.297	5.328	0.616	0.000	0.000
12.6	6.59	0.289	0.217	0.072	0.000	0.000	0.000	0.000
22.5	6.60	0.069	0.069	0.000	0.000	0.000	0.000	0.000
3.8	6.80	18.602	1.646	6.184	8.613	2.159	0.000	0.000
13.1	6.79	0.277	0.202	0.075	0.000	0.000	0.000	0.000
22.3	6.79	0.111	0.111	0.000	0.000	0.000	0.000	0.000
3.6	6.98	26.074	2.166	9.005	11.877	3.003	0.023	0.000
13.0	6.98	0.187	0.124	0.062	0.000	0.000	0.000	0.000
22.6	7.04	0.297	0.279	0.019	0.000	0.000	0.000	0.000
31.9	7.02	0.070	0.070	0.000	0.000	0.000	0.000	0.000
3.2	7.16	11.206	0.801	3.891	5.180	1.292	0.042	0.000
20.6	7.20	0.415	0.355	0.060	0.000	0.000	0.000	0.000
2.4	7.42	9.402	0.726	3.911	4.342	0.418	0.005	0.000
20.3	7.36	0.518	0.381	0.137	0.000	0.000	0.000	0.000
2.3	7.61	1.702	0.130	0.705	0.800	0.068	0.000	0.000
20.2	7.60	0.348	0.283	0.065	0.000	0.000	0.000	0.000
34.9	7.59	0.056	0.055	0.001	0.000	0.000	0.000	0.000
2.3	7.71	0.589	0.044	0.241	0.279	0.025	0.000	0.000
20.2	7.75	0.553	0.307	0.246	0.000	0.000	0.000	0.000
35.8	7.77	0.099	0.098	0.001	0.000	0.000	0.000	0.000
20.2	7.91	0.086	0.047	0.039	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 100.0
Mean src-site R= 5.0 km; M= 6.81; eps0= 0.44. Mean calculated for all sources.
Modal src-site R= 3.6 km; M= 6.98; eps0= 0.10 from peak (R,M) bin
MODE R*= 3.1km; M*= 6.98; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 11.877

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	48.32	4.2	7.11	0.24
California B-faults GR	33.00	3.9	6.82	0.20
CA Compr. crustal gridded	18.34	8.3	5.99	1.37

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Newport-Inglewood (Offshore) Cha	4.51	8.9	6.89	1.48	157.6
Newport-Inglewood, alt 1 Char	4.77	2.4	7.14	0.41	-128.5
Newport-Inglewood, alt 2 Char	4.71	2.2	7.14	0.43	-129.1
San Joaquin Hills Char	23.83	3.3	6.97	-0.26	114.4
Newport Inglewood Connected alt	4.17	2.4	7.50	0.37	-128.5
Newport Inglewood Connected alt	4.22	2.2	7.50	0.36	-129.1
San Joaquin Hills GR	22.13	3.7	6.75	-0.06	114.0

Newport-Inglewood, alt 1 GR	2.29	4.0	6.87	0.73	-105.4
Newport-Inglewood, alt 2 GR	2.44	3.8	6.87	0.73	-109.7
Newport Inglewood Connected alt1	2.73	3.2	7.05	0.58	-130.5
Newport Inglewood Connected alt2	2.77	3.1	7.05	0.57	-131.1

#*********End of deaggregation corresponding to Mean Hazard w/all GMPEs *********#

PSHA Deaggregation. %contributions. site: Le_Bard_Park long: 117.951 W., lat: 33.665 N.
Vs30(m/s)= 760.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.6496 g. Weight * Computed_Rate_Ex 0.952E-04
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00246
#This deaggregation corresponds to Boore-Atkinson 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
7.7	5.21	0.014	0.014	0.000	0.000	0.000	0.000	0.000
8.0	5.41	0.033	0.033	0.000	0.000	0.000	0.000	0.000
8.1	5.61	0.063	0.063	0.000	0.000	0.000	0.000	0.000
8.2	5.80	0.097	0.097	0.000	0.000	0.000	0.000	0.000
7.0	6.02	0.219	0.208	0.011	0.000	0.000	0.000	0.000
6.5	6.20	0.332	0.296	0.036	0.000	0.000	0.000	0.000
12.8	6.21	0.056	0.056	0.000	0.000	0.000	0.000	0.000
6.7	6.40	0.357	0.280	0.077	0.000	0.000	0.000	0.000
12.9	6.40	0.096	0.096	0.000	0.000	0.000	0.000	0.000
4.4	6.62	2.408	0.468	1.322	0.618	0.000	0.000	0.000
13.2	6.60	0.056	0.056	0.000	0.000	0.000	0.000	0.000
21.9	6.62	0.022	0.022	0.000	0.000	0.000	0.000	0.000
3.9	6.80	4.529	0.596	1.894	2.006	0.033	0.000	0.000
13.5	6.79	0.076	0.069	0.007	0.000	0.000	0.000	0.000
21.9	6.80	0.056	0.056	0.000	0.000	0.000	0.000	0.000
32.1	6.79	0.018	0.018	0.000	0.000	0.000	0.000	0.000
3.6	6.99	6.889	0.743	2.942	3.025	0.179	0.000	0.000
13.2	6.98	0.060	0.046	0.013	0.000	0.000	0.000	0.000
21.3	7.03	0.144	0.128	0.016	0.000	0.000	0.000	0.000
31.9	7.02	0.068	0.068	0.000	0.000	0.000	0.000	0.000
3.2	7.18	2.777	0.257	1.217	1.269	0.035	0.000	0.000
20.8	7.19	0.235	0.187	0.049	0.000	0.000	0.000	0.000
34.4	7.20	0.042	0.042	0.000	0.000	0.000	0.000	0.000
2.4	7.42	2.697	0.221	1.225	1.223	0.029	0.000	0.000
20.3	7.35	0.272	0.186	0.086	0.000	0.000	0.000	0.000
36.0	7.36	0.030	0.030	0.000	0.000	0.000	0.000	0.000
43.0	7.36	0.013	0.013	0.000	0.000	0.000	0.000	0.000
2.3	7.60	0.589	0.062	0.282	0.238	0.008	0.000	0.000
20.2	7.58	0.167	0.125	0.042	0.000	0.000	0.000	0.000
34.8	7.59	0.044	0.043	0.001	0.000	0.000	0.000	0.000
2.3	7.71	0.181	0.015	0.080	0.084	0.004	0.000	0.000
20.2	7.75	0.288	0.158	0.130	0.000	0.000	0.000	0.000
36.1	7.77	0.084	0.082	0.003	0.000	0.000	0.000	0.000
20.2	7.91	0.040	0.019	0.021	0.000	0.000	0.000	0.000
35.0	7.94	0.012	0.011	0.001	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 23.1
Mean src-site R= 5.2 km; M= 6.99; eps0= 0.70. Mean calculated for all sources.
Modal src-site R= 3.6 km; M= 6.99; eps0= 0.45 from peak (R,M) bin
MODE R*= 3.1km; M*= 6.99; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 3.025

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	13.27	4.8	7.14	0.59
California B-faults GR	7.92	4.0	6.86	0.61

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Newport-Inglewood (Offshore) Cha	1.28	8.9	6.90	1.56	157.6
Newport-Inglewood, alt 1 Char	1.52	2.4	7.14	0.46	-128.5
Newport-Inglewood, alt 2 Char	1.40	2.2	7.15	0.54	-129.1

San Joaquin Hills Char	5.47	3.3	6.97	0.24	114.4
Newport Inglewood Connected alt	1.28	2.4	7.50	0.46	-128.5
Newport Inglewood Connected alt	1.28	2.2	7.50	0.46	-129.1
San Joaquin Hills GR	4.66	3.6	6.77	0.43	114.0
Newport-Inglewood, alt 1 GR	0.69	3.9	6.88	0.80	-105.4
Newport-Inglewood, alt 2 GR	0.69	3.8	6.88	0.86	-109.7
Newport Inglewood Connected alt1	0.81	3.2	7.07	0.68	-130.5
Newport Inglewood Connected alt2	0.81	3.1	7.06	0.69	-131.1

#*********End of deaggregation corresponding to Boore-Atkinson 2008 *********#

PSHA Deaggregation. %contributions. site: Le_Bard_Park long: 117.951 W., lat: 33.665 N.
Vs30(m/s)= 760.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.6496 g. Weight * Computed_Rate_Ex 0.148E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.01202
#This deaggregation corresponds to Campbell-Bozorgnia 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
8.1	5.05	0.092	0.092	0.000	0.000	0.000	0.000	0.000
8.2	5.21	0.301	0.301	0.000	0.000	0.000	0.000	0.000
8.3	5.41	0.502	0.477	0.025	0.000	0.000	0.000	0.000
8.4	5.60	0.593	0.503	0.090	0.000	0.000	0.000	0.000
8.4	5.80	0.547	0.437	0.110	0.000	0.000	0.000	0.000
7.3	6.01	0.660	0.471	0.189	0.000	0.000	0.000	0.000
13.8	6.03	0.078	0.078	0.000	0.000	0.000	0.000	0.000
6.6	6.20	0.826	0.465	0.361	0.000	0.000	0.000	0.000
12.7	6.20	0.217	0.217	0.000	0.000	0.000	0.000	0.000
6.9	6.40	0.896	0.389	0.507	0.000	0.000	0.000	0.000
12.7	6.39	0.275	0.271	0.004	0.000	0.000	0.000	0.000
4.5	6.61	5.311	0.531	1.951	2.357	0.472	0.000	0.000
12.4	6.59	0.095	0.088	0.007	0.000	0.000	0.000	0.000
23.1	6.59	0.027	0.027	0.000	0.000	0.000	0.000	0.000
3.7	6.81	8.214	0.637	2.221	3.553	1.803	0.000	0.000
13.1	6.80	0.076	0.069	0.007	0.000	0.000	0.000	0.000
23.2	6.77	0.024	0.024	0.000	0.000	0.000	0.000	0.000
3.6	6.98	8.852	0.704	2.768	3.918	1.439	0.023	0.000
13.0	6.97	0.049	0.041	0.008	0.000	0.000	0.000	0.000
23.3	7.03	0.052	0.051	0.000	0.000	0.000	0.000	0.000
3.2	7.15	4.296	0.277	1.337	1.911	0.731	0.040	0.000
20.8	7.20	0.058	0.057	0.001	0.000	0.000	0.000	0.000
2.4	7.41	2.899	0.243	1.272	1.270	0.111	0.003	0.000
20.3	7.36	0.078	0.078	0.000	0.000	0.000	0.000	0.000
2.3	7.61	0.497	0.043	0.226	0.217	0.010	0.000	0.000
20.2	7.58	0.026	0.025	0.001	0.000	0.000	0.000	0.000
2.3	7.71	0.170	0.015	0.077	0.074	0.004	0.000	0.000
20.2	7.74	0.078	0.066	0.012	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 35.9
Mean src-site R= 4.5 km; M= 6.81; eps0= 0.21. Mean calculated for all sources.
Modal src-site R= 3.6 km; M= 6.98; eps0= -0.06 from peak (R,M) bin
MODE R*= 3.1km; M*= 6.98; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 3.918

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	17.14	3.8	7.07	0.01
California B-faults GR	12.81	3.7	6.80	-0.04
CA Compr. crustal gridded	5.90	8.1	6.06	1.37

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Newport-Inglewood (Offshore) Cha	1.47	8.9	6.89	1.49	157.6
Newport-Inglewood, alt 1 Char	1.47	2.4	7.14	0.47	-128.5
Newport-Inglewood, alt 2 Char	1.48	2.2	7.14	0.47	-129.1
San Joaquin Hills Char	9.90	3.3	6.97	-0.54	114.4
Newport Inglewood Connected alt	1.23	2.4	7.50	0.46	-128.5

Newport Inglewood Connected alt	1.25	2.2	7.50	0.45	-129.1
San Joaquin Hills GR	9.53	3.7	6.75	-0.31	114.0
Newport-Inglewood, alt 1 GR	0.71	3.9	6.87	0.77	-105.4
Newport-Inglewood, alt 2 GR	0.76	3.7	6.87	0.76	-109.7
Newport Inglewood Connected alt1	0.84	3.1	7.05	0.63	-130.5
Newport Inglewood Connected alt2	0.85	3.0	7.05	0.62	-131.1

#*********End of deaggregation corresponding to Campbell-Bozorgnia 2008 *********#

PSHA Deaggregation. %contributions. site: Le_Bard_Park long: 117.951 W., lat: 33.665 N.
Vs30(m/s)= 760.0 (some WUS atten. models use Site Class not Vs30).
NSHMP 2007-08 See USGS OFR 2008-1128. dM=0.2 below
Return period: 2475 yrs. Exceedance PGA =0.6496 g. Weight * Computed_Rate_Ex 0.169E-03
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00945
#This deaggregation corresponds to Chiou-Youngs 2008

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
8.3	5.05	0.489	0.486	0.003	0.000	0.000	0.000	0.000
8.3	5.20	1.055	0.990	0.065	0.000	0.000	0.000	0.000
8.4	5.40	1.089	0.946	0.143	0.000	0.000	0.000	0.000
15.8	5.41	0.028	0.028	0.000	0.000	0.000	0.000	0.000
8.4	5.60	1.064	0.855	0.209	0.000	0.000	0.000	0.000
15.9	5.61	0.059	0.059	0.000	0.000	0.000	0.000	0.000
8.4	5.80	0.998	0.717	0.281	0.000	0.000	0.000	0.000
16.0	5.80	0.087	0.087	0.000	0.000	0.000	0.000	0.000
7.3	6.01	1.186	0.621	0.565	0.000	0.000	0.000	0.000
14.2	6.03	0.232	0.232	0.000	0.000	0.000	0.000	0.000
6.6	6.20	1.301	0.496	0.805	0.000	0.000	0.000	0.000
13.0	6.20	0.510	0.495	0.016	0.000	0.000	0.000	0.000
7.0	6.40	1.242	0.419	0.823	0.000	0.000	0.000	0.000
12.9	6.39	0.561	0.501	0.060	0.000	0.000	0.000	0.000
4.6	6.61	4.746	0.532	1.984	2.102	0.127	0.000	0.000
12.4	6.59	0.138	0.109	0.029	0.000	0.000	0.000	0.000
3.9	6.80	6.737	0.627	2.237	3.242	0.631	0.000	0.000
12.9	6.79	0.125	0.093	0.031	0.000	0.000	0.000	0.000
22.3	6.79	0.030	0.030	0.000	0.000	0.000	0.000	0.000
3.6	6.98	9.585	0.752	3.174	4.565	1.095	0.000	0.000
12.8	6.97	0.079	0.055	0.023	0.000	0.000	0.000	0.000
23.8	7.02	0.069	0.069	0.001	0.000	0.000	0.000	0.000
3.3	7.17	4.133	0.267	1.337	2.000	0.527	0.003	0.000
21.1	7.19	0.142	0.130	0.012	0.000	0.000	0.000	0.000
2.4	7.42	3.639	0.225	1.286	1.850	0.278	0.001	0.000
20.3	7.35	0.160	0.120	0.040	0.000	0.000	0.000	0.000
2.3	7.60	0.782	0.062	0.326	0.345	0.049	0.000	0.000
20.2	7.56	0.103	0.071	0.032	0.000	0.000	0.000	0.000
2.3	7.71	0.238	0.015	0.084	0.121	0.018	0.000	0.000
20.2	7.74	0.259	0.155	0.104	0.000	0.000	0.000	0.000
20.2	7.91	0.036	0.020	0.016	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 41.0
Mean src-site R= 5.3 km; M= 6.71; eps0= 0.49. Mean calculated for all sources.
Modal src-site R= 3.6 km; M= 6.98; eps0= 0.07 from peak (R,M) bin
MODE R*= 3.1km; M*= 6.98; EPS.INTERVAL: 0 to 1 sigma % CONTRIB.= 4.565

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
California B-faults Char	17.90	4.2	7.12	0.19
California B-faults GR	12.26	3.9	6.82	0.19
CA Compr. crustal gridded	10.82	8.5	5.92	1.32

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Newport-Inglewood (Offshore) Cha	1.75	8.9	6.90	1.41	157.6
Newport-Inglewood, alt 1 Char	1.77	2.4	7.15	0.32	-128.5
Newport-Inglewood, alt 2 Char	1.84	2.2	7.15	0.30	-129.1
San Joaquin Hills Char	8.46	3.3	6.97	-0.27	114.4

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Newport Inglewood Connected alt     1.65     2.4   7.50    0.23    -128.5
Newport Inglewood Connected alt     1.69     2.2   7.50    0.21    -129.1
San Joaquin Hills GR                7.94     3.7   6.75   -0.04     114.0
Newport-Inglewood, alt 1 GR         0.89     4.1   6.86    0.65    -105.4
Newport-Inglewood, alt 2 GR         0.99     3.9   6.86    0.62    -109.7
Newport Inglewood Connected alt1    1.08     3.3   7.05    0.47    -130.5
Newport Inglewood Connected alt2    1.11     3.1   7.05    0.45    -131.1
#*********End of deaggregation corresponding to Chiou-Youngs 2008        *********#

******************** Southern California ****************************************
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APPENDIX E

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

1.0 General

1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 Geotechnical Consultant: Prior to commencement of work, the owner shall employ a geotechnical consultant. The geotechnical consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing: Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed

immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching: Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas: All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General: Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import: If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers: Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning: Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

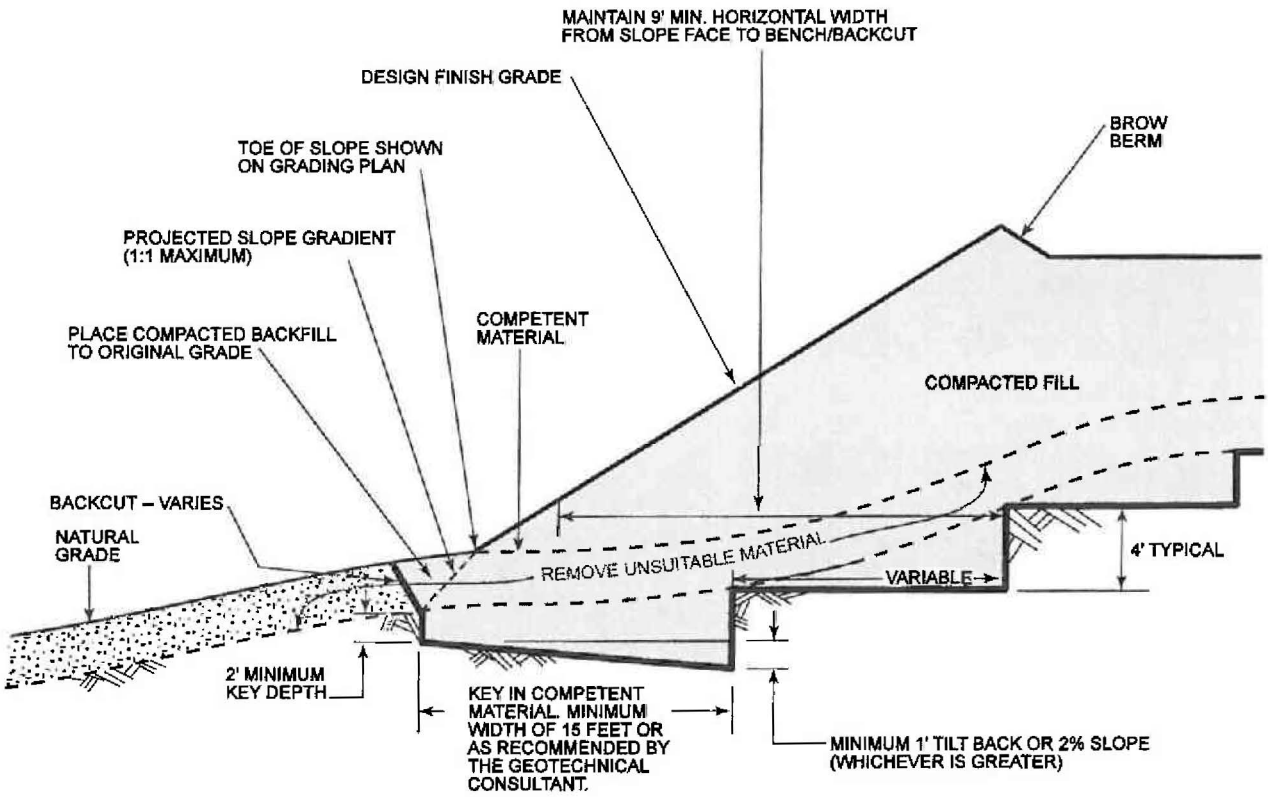
7.2 Bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum 90 percent of maximum from 1 foot above the top of the conduit to the surface, except in traveled ways (see Section 7.6 below).

7.3 Jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

7.4 Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.5 Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

7.6 Trench backfill in the upper foot measured from finish grade within existing or future traveled way, shoulder, and other paved areas (or areas to receive pavement) should be placed to a minimum 95 percent relative compaction.



NOTE: BENCHING SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE SOIL ENGINEER. WHERE THE NATURAL SLOPE APPROACHES OR EXCEEDS THE DESIGN SLOPE RATIO, SPECIAL RECOMMENDATIONS WILL BE PROVIDED BY THE GEOTECHNICAL ENGINEER.

FIGURE 1

TYPICAL FILL KEY ABOVE NATURAL SLOPE MINIMUM STANDARD GRADING DETAILS

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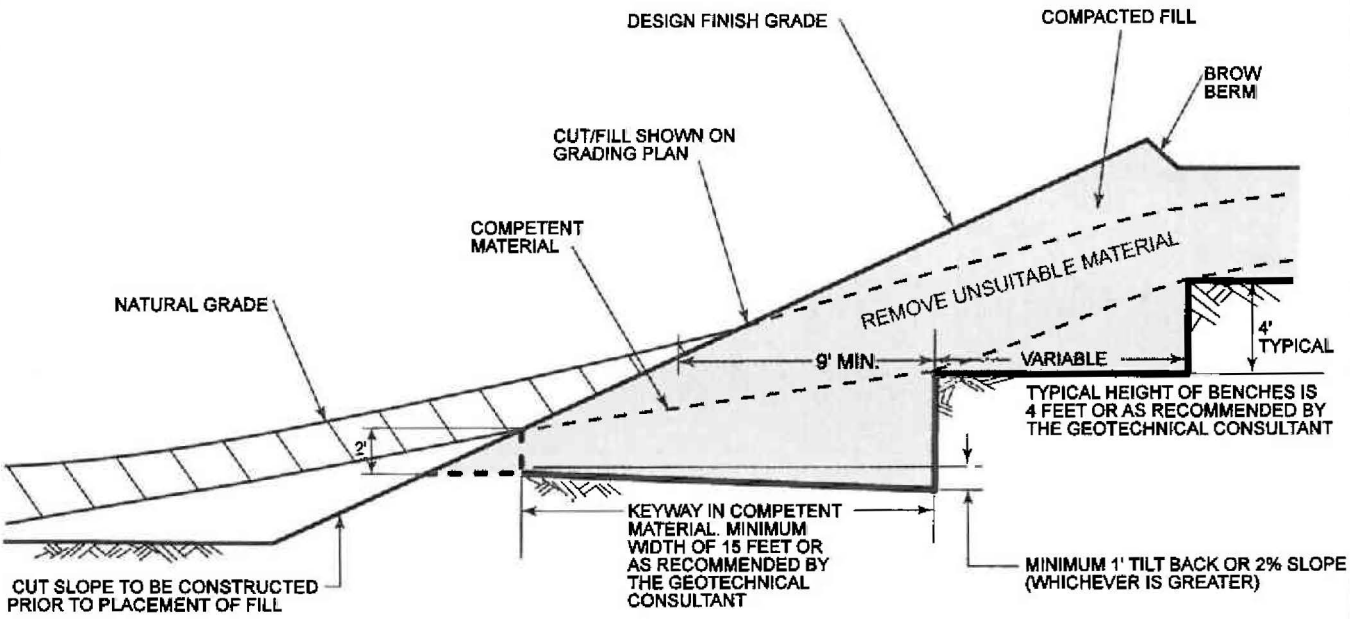


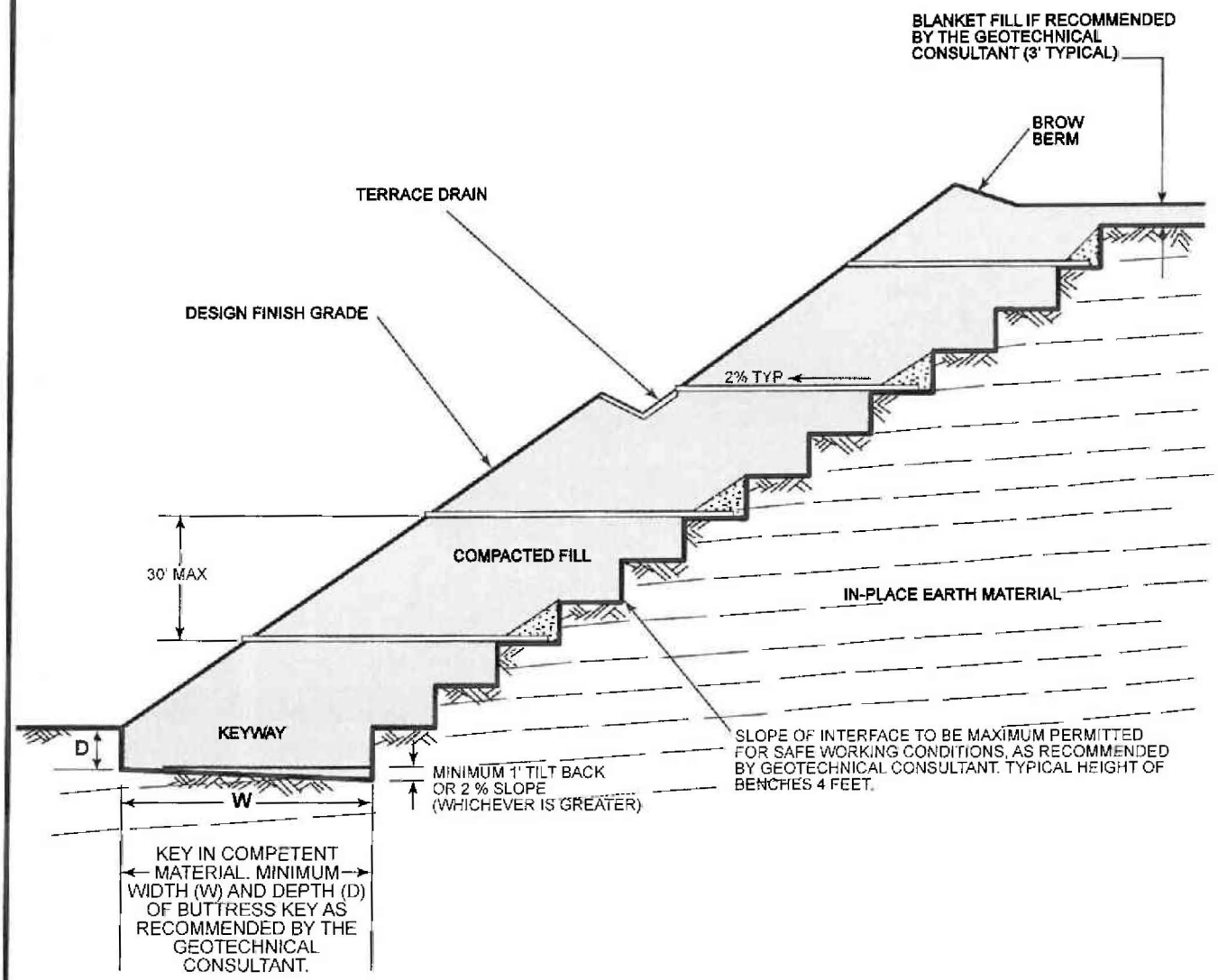
NOTE: THE FILL PORTION OF THE SLOPE SHALL BE COMPACTED AS STATED IN THE PROJECT SPECIFICATIONS.

FIGURE 2

TYPICAL FILL ABOVE CUT SLOPE
MINIMUM STANDARD GRADING DETAILS

NMG Geotechnical, Inc.

8/03 TYP FILL ABOVE CUT SLOPE.sl



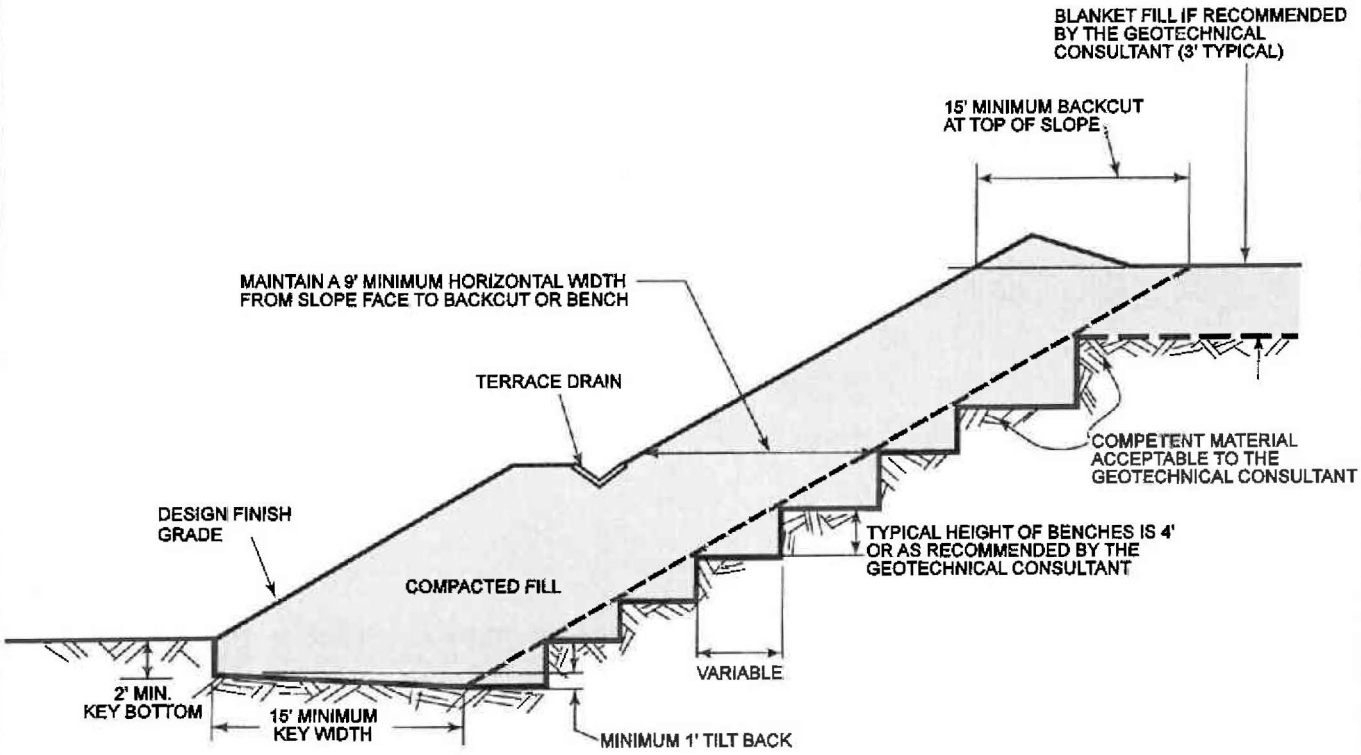
NOTE: SUBDRAIN DETAILS, SEE FIGURE 5.

FIGURE 3

TYPICAL BUTTRESS FILL
MINIMUM STANDARD GRADING DETAILS

NMG
Geotechnical, Inc.

1/04 TYP BUTTRESS FILL.ai



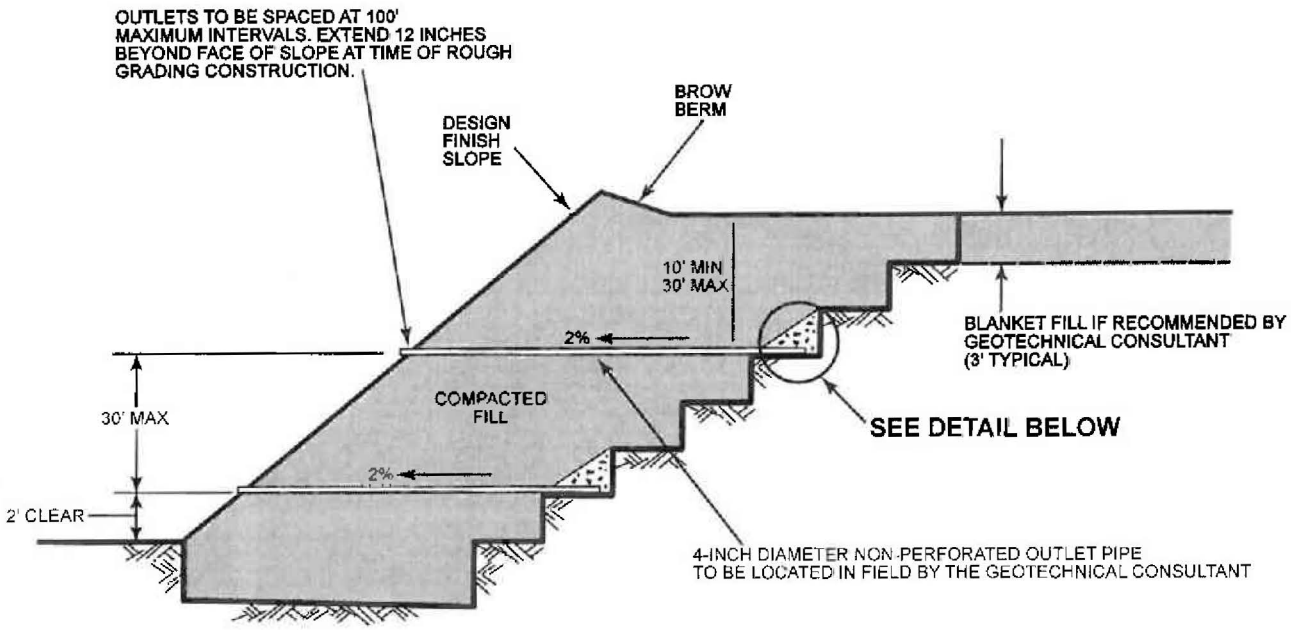
NOTE:
SEE FIGURE 5 FOR TYPICAL SUBDRAIN DETAILS FOR STABILIZATION FILLS

FIGURE 4

TYPICAL STABILIZATION FILL MINIMUM STANDARD GRADING DETAILS

NMG Geotechnical, Inc.

9/96 STABILIZATION FILL.ai



FILTER MATERIAL - MINIMUM OF THREE CUBIC FEET PER FOOT OF PIPE. SEE FILTER MATERIAL SPECIFICATION.

ALTERNATE: IN LIEU OF FILTER MATERIAL, THREE CUBIC FEET OF GRAVEL PER FOOT OF SUBDRAIN (WITHOUT PIPE) MAY BE ENCASED IN FILTER FABRIC. SEE GRAVEL SPECIFICATION, AND FIGURE 6 FOR FILTER FABRIC SPECIFICATION

"GRAVEL" TO CONSIST OF 1/2" TO 1" CRUSHED ROCK PER STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION.

FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

DETAIL

OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW

"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT.

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

MINIMUM 4-INCH DIAMETER SCHEDULE 40 ASTM D1527 OR D1785 OR SDR 35 ASTM D2751 OR D 3034. FOR FILL DEPTH OF 90 FEET OR GREATER, USE ONLY SCHEDULE 40 OR EQUIVALENT. THERE SHALL BE A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

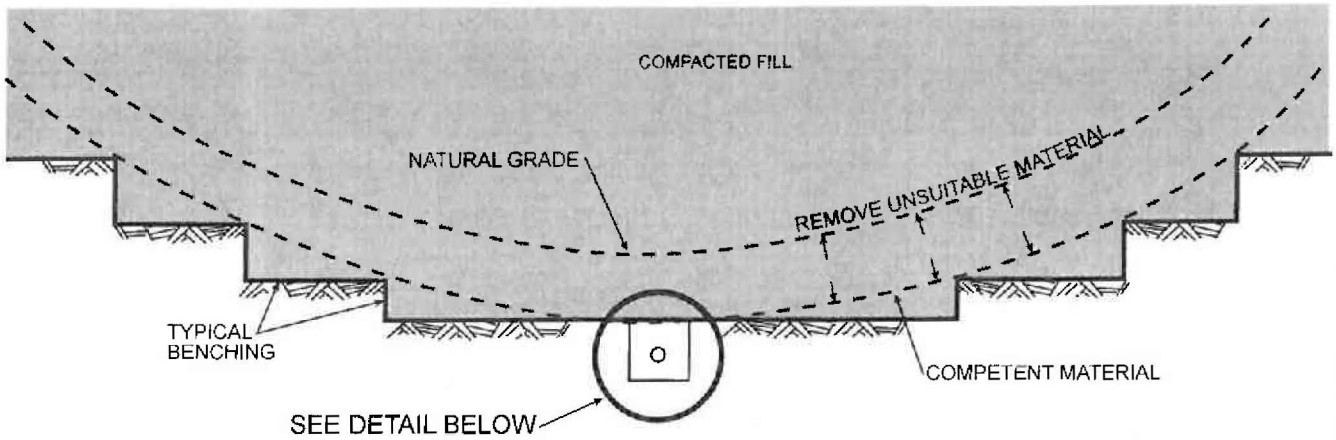
NOTE:
TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

FIGURE 5

TYPICAL STABILIZATION AND BUTTRESS FILL SUBDRAINS MINIMUM STANDARD GRADING DETAILS

NMG Geotechnical, Inc.

8/96 STAB. BUTTRESS FILL SUBDRAINS.ai



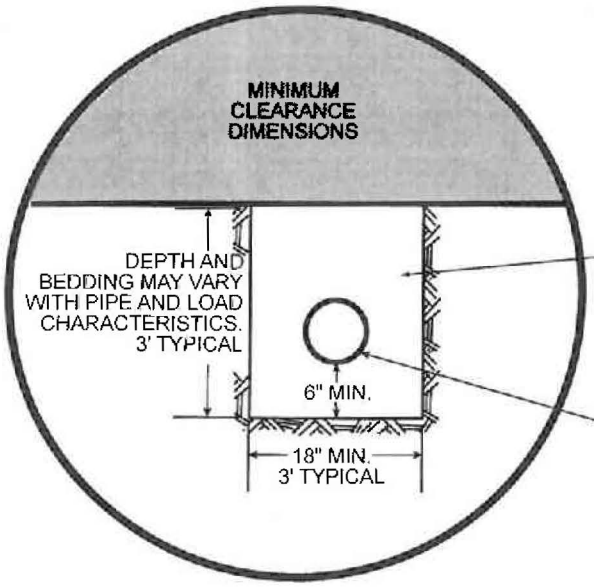
FILTER FABRICS SHALL BE PERMEABLE NON-WOVEN POLYESTER, NYLON, OR POLYPROPYLENE MATERIAL CONFORMING TO THE FOLLOWING:

1) GRAB TENSILE STRENGTH, POUNDS, MIN. ASTM D 4632.....90
2) ELONGATION, AT PEAK LOAD, PERCENT, MIN. ASTM D 4632.....50
3) PUNCTURE STRENGTH, LBS., MIN. ASTM D 3787.....45
4) COEFFICIENT OF WATER PERMITTIVITY, 1/SEC. ASTM D 4491.....>0.7
5) BURST STRENGTH, P.S.I., MIN. ASTM D 3786.....180

NOTES: DOWNSTREAM 20' OF PIPE AT OUTLET SHALL BE NON-PERFORATED AND BACKFILLED WITH FINE-GRAINED MATERIAL

PIPE SHALL BE A MINIMUM OF 4-INCH DIAMETER. FOR RUNS OF 500 FEET OR MORE, USE 6-INCH DIAMETER PIPE, OR AS RECOMMENDED BY THE GEOTECHNICAL CONSULTANT

DETAIL



FILTER MATERIAL - MINIMUM OF NINE CUBIC FEET PER FOOT OF PIPE, SEE FIGURE 5 FOR FILTER MATERIAL SPECIFICATIONS.

ALTERNATE: IN LIEU OF FILTER MATERIAL, NINE CUBIC FEET OF GRAVEL PER FOOT OF SUBDRAIN (WITHOUT PIPE) MAY BE ENCASED IN FILTER FABRIC. SEE FIGURE 5 TO GRAVEL SPECIFICATION. SEE ABOVE FOR FILTER FABRIC SPECIFICATION. FILTER FABRIC SHALL BE LAPPED MINIMUM OF 12 INCHES ON ALL JOINTS.

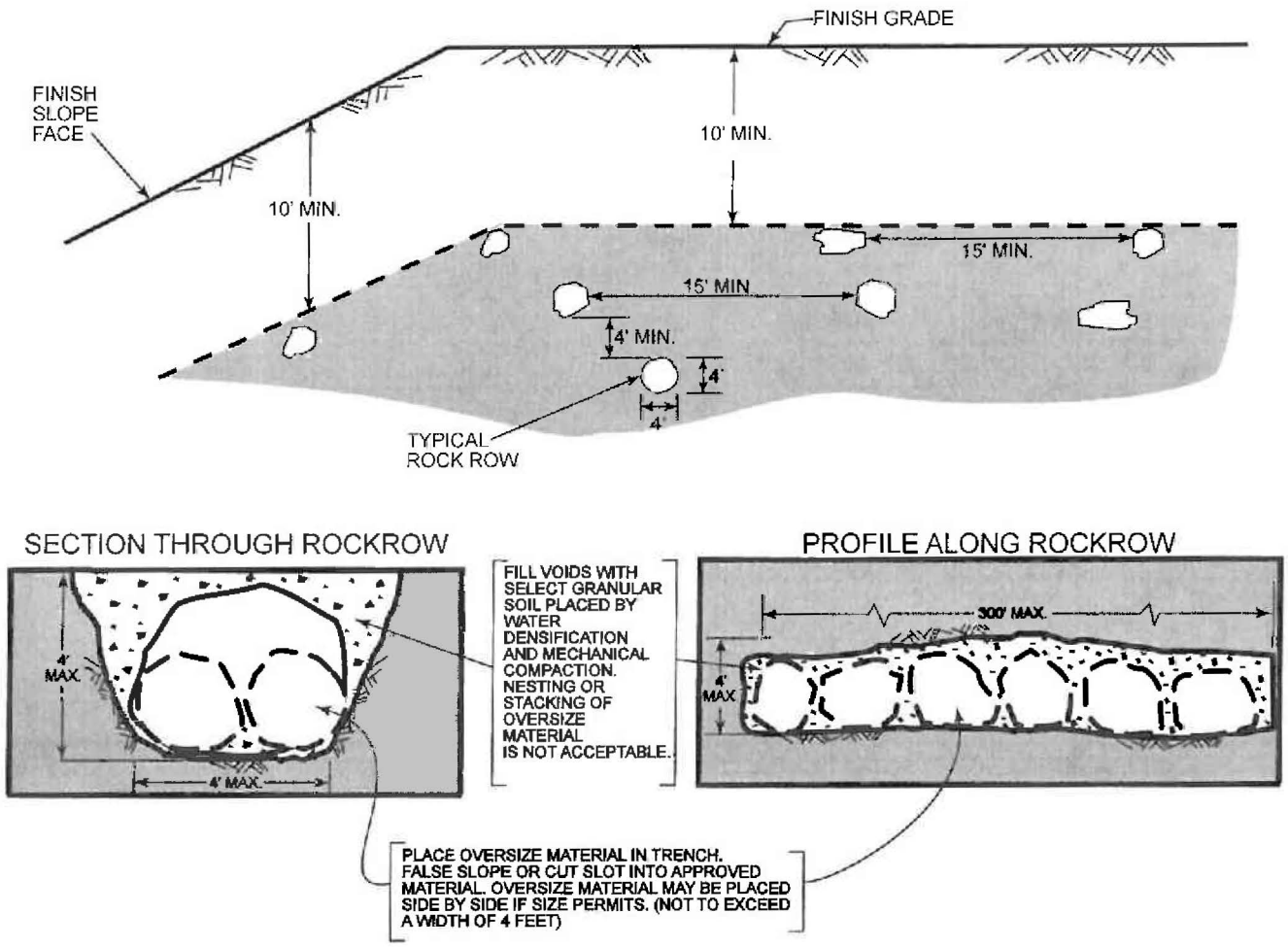
MINIMUM 4 INCH DIAMETER SCHEDULE 40 ASTM D 1527, OR D 1785, OR SDR 35 ASTM 2751 OR D 3034. FOR FILL DEPTH OF 90 FEET OR GREATER, USE ONLY SCHEDULE 40 OR APPROVED EQUIVALENT. THERE SHALL BE A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE.

FIGURE 6

TYPICAL CANYON SUBDRAIN MINIMUM STANDARD GRADING DETAILS

NMG Geotechnical, Inc.

Rev. 8/96 CANYON SUBDRAIN.ai



NOTES:

A) OVERSIZED ROCK IS DEFINED AS LARGER THAN 12" IN SIZE (IN GREATEST DIMENSION).

B) SPACE BETWEEN ROCKROWS SHOULD BE ONE EQUIPMENT WIDTH OR A MINIMUM OF 15 FEET.

C) THE WIDTH AND HEIGHT OF THE ROCKROW SHALL BE LIMITED TO FOUR FEET AND THE LENGTH LIMITED TO 300 FEET UNLESS APPROVED OTHERWISE BY THE GEOTECHNICAL CONSULTANT. OVERSIZE SHOULD BE PLACED WITH FLATEST SIDE ON THE BOTTOM.

D) OVERSIZE MATERIAL EXCEEDING FOUR FEET MAY BE PLACED ON AN INDIVIDUAL BASIS IF APPROVED BY THE GEOTECHNICAL CONSULTANT.

E) FILLING OF VOIDS WILL REQUIRE SELECT GRANULAR SOIL (SE > 20, OR LESS THAN 20 PERCENT FINES) AS APPROVED BY THE GEOTECHNICAL CONSULTANT. VOIDS IN THE ROCKROW TO BE FILLED BY WATER DENSIFYING GRANULAR SOIL INTO PLACE ALONG WITH MECHANICAL COMPACTION EFFORT.

F) IF APPROVED BY THE GEOTECHNICAL CONSULTANT, ROCKROWS MAY BE PLACED DIRECTLY ON COMPETENT MATERIALS OR BEDROCK, PROVIDED ADEQUATE SPACE IS AVAILABLE FOR COMPACTION.

G) THE FIRST LIFT OF MATERIAL ABOVE THE ROCKROW SHALL CONSIST OF GRANULAR MATERIAL AND SHALL BE PROOF-ROLLED WITH A D-8 OR LARGER DOZER OR EQUIVALENT.

H) ROCKROWS NEAR SLOPES SHOULD BE ORIENTED PARALLEL TO SLOPE FACE.

I) NESTING OR STACKING OF ROCKS IS NOT ACCEPTABLE.

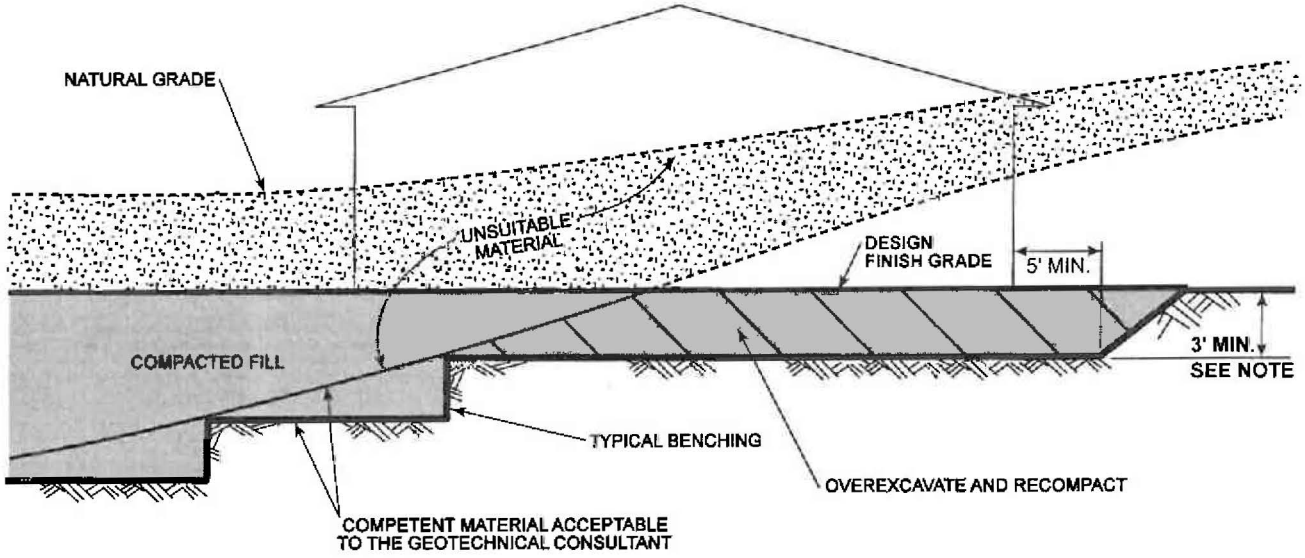
FIGURE 7

TYPICAL OVERSIZE ROCK PLACEMENT METHOD MINIMUM STANDARD GRADING DETAIL FOR STRUCTURAL FILL

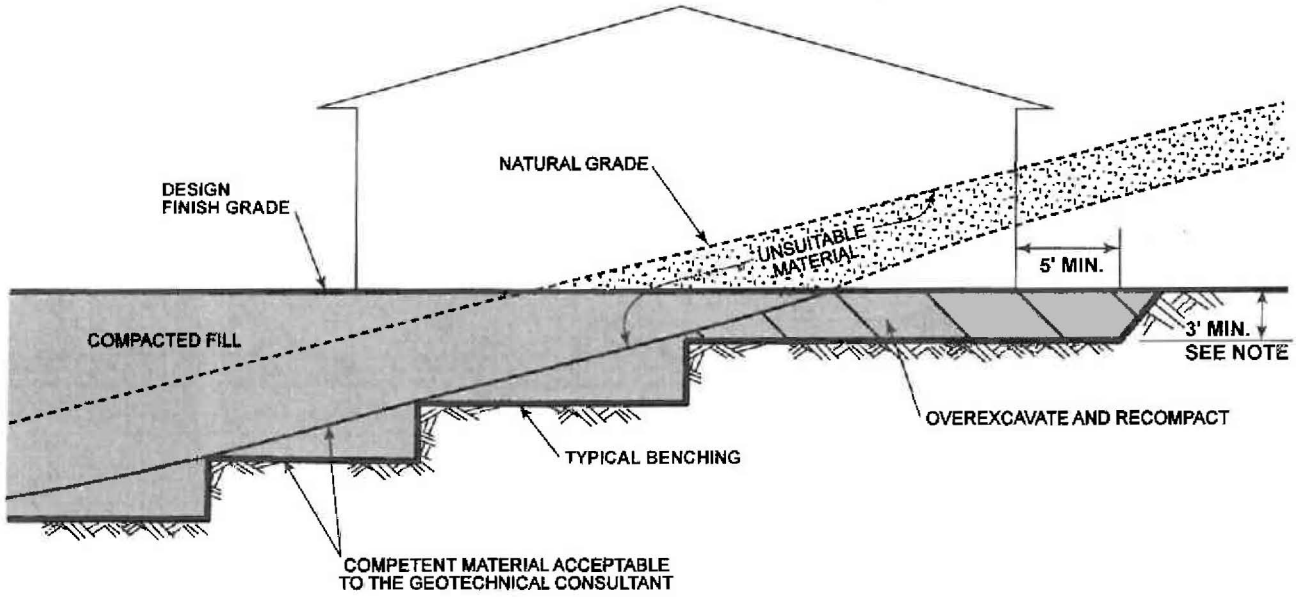
NMG Geotechnical, Inc.

3/04 TYP OVERSIZE ROCK PLACEMENT.ai

CUT LOT



CUT FILL LOT (TRANSITION)



NOTE: DEEPER THAN THE 3-FOOT OVEREXCAVATION MAY BE RECOMMENDED BY THE GEOTECHNICAL CONSULTANT IN STEEP TRANSITIONS.

FIGURE 8

TYPICAL OVEREXCAVATION OF DAYLIGHT LINE MINIMUM STANDARD GRADING DETAILS

NMG Geotechnical, Inc.