Appendix D
Geotechnical Evaluation
Appendix

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GEOTECHNICAL EVALUATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

SERRANO II PROJECT

CITY OF CLAREMONT, LOS ANGELES COUNTY, CALIFORNIA

PREPARED FOR

D•R•HORTON LOS ANGELES HOLDING COMPANY, INC.
2280 WARDLOW CIRCLE, SUITE 100
CORONA, CALIFORNIA 92880

PREPARED BY

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PROJECT NO. 1034-CR3

AUGUST 12, 2013
August 12, 2013
Project No. 1034-CR3

D-R Horton Los Angeles Holding Company, Inc.
2280 Wardlow Circle, Suite 100
Corona, California 92880

Attention: Mr. Pat Potts

Subject: Geotechnical Evaluation
       Proposed Residential Development
       Serrano II Project
       City of Claremont, Los Angeles County, California

Dear Mr. Potts:

We are pleased to provide herein the results of our geotechnical evaluation for the subject property located in the City of Claremont, County of Los Angeles, California. This report presents a discussion of our evaluation and provides preliminary geotechnical recommendations for earthwork, foundation design, and construction. In our opinion, site development appears feasible from a geotechnical viewpoint provided that the recommendations included herein are incorporated into the design and construction phases of site development.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to call our office.

Respectfully submitted,
GeoTek, Inc.

Jeffrey M. Pflueger
CEG 2499, Exp. 07/31/14
Project Geologist

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GE 2524, Exp. 06/30/15
Senior Project Engineer

Distribution: (1) Addressee via email
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Figure 1 – Site Location & Topographic Map
Figure 2 – Exploration Location Map
Figure 3 – Geologic Map

Appendix A – Logs of Exploratory Trenches
Appendix B – Laboratory Testing Results
Appendix C – General Grading Guidelines
1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical conditions in the area of proposed construction. Services provided for this study included the following:

- Research and review of available geologic data and general information pertinent to the site,
- Site reconnaissance,
- Excavation of five (5) exploratory test trenches onsite,
- Collection of bulk soil samples of the onsite materials,
- Laboratory testing of the soil samples collected from the site,
- Provide samples to GeoTek's sub-consultant for a soil corrosivity study,
- Review and evaluation of site seismicity, and
- Compilation of this geotechnical report which presents our recommendations for site development.

The intent of this report is to aid in the evaluation of the site for future development from a geotechnical perspective. The professional opinions and geotechnical information contained in this report will likely need to be updated based on our review of final site development plans. These should be provided to GeoTek for review when available.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The subject Serrano II Project is located along the southern side of W. Baseline Road between N. Mountain Avenue and N. Indian Hill Boulevard in the City of Claremont, Los Angeles County, California (see Figures 1 and 2). The rectangular shaped property is comprised of roughly 3.5 acres of land. Topography across the site slopes gently down toward the southwest, with a total relief on the order of roughly 15 feet.
Mixed-use properties immediately surround the subject site including vacant buildings and parking lot areas to the west, Interstate 210 Freeway to the south, commercial property to the east, and residential properties to the north of W. Baseline Road. The property located to the west of the site is currently proposed for residential development (Serrano I) and was previously evaluated as a part of the overall project (GeoTek, 2012, 2013a, and 2013b).

The subject property is currently occupied by the Claremont Unified School District and is being used as a service center and included offices and maintenance/repair facilities. There are currently two (2) large buildings on the property and several storage bins, maintenance equipment, and supplies are situated primarily throughout the eastern portion of the project site. The central and western portions of the project site are paved and serve primarily as access and parking areas (see Figure 2). Several large trees are located primarily along the perimeter of the property. An existing block wall, which may be retaining a small amount of material on the north side, is located adjacent to the south side of the property. This wall is understood to have been constructed as part of Interstate 210 improvements, and is owned by Caltrans.

2.2 PROPOSED DEVELOPMENT

It is our understanding that proposed site improvements include razing all of the existing site improvements, and constructing 38 new detached single-family residences (Serrano Phase II). No grading plans or specific information has yet been provided for the proposed improvements. A site plan titled “Alt 5B Site Study” prepared by William Hezmalhalch Architects, Inc., dated June 21, 2013 was provided that shows a conceptual site plan indicating a layout for the proposed new residences. The plan also shows 54 proposed new detached single-family residences to the west of the subject project site (Serrano Phase I). It is assumed that the proposed new residences will likely be two-story structures, with no basements or below-ground parking. It is also assumed that the new structures will utilize wood-frames and conventional foundation systems with slab-on-grade construction. Associated small retaining walls (six [6] feet high or less), driveways, flatwork and landscaping are also anticipated. Structural loads are anticipated to be typical for this type of construction.

If site development differs from the assumptions made herein, the recommendations included in this report should be subject to further review and evaluation. Site development plans should be reviewed by GeoTek when they become available.
3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION

Our recent field exploration was conducted on July 24, 2013. An engineering geologist from GeoTek logged five (5) exploratory trenches, excavated by a backhoe. The trenches were located at various locations across the site (see Exploration Location Map, Figure 2). The approximate depths of the trenches were up to 4.5 feet. Logs of the exploratory trenches are included in Appendix A. GeoTek collected samples of onsite soil materials encountered in the excavations.

3.2 LABORATORY TESTING

Laboratory testing was performed on bulk soil samples collected during the field exploration. The purpose of the laboratory testing was to confirm the field classification of the soil materials encountered and to evaluate the soils physical properties for use in the engineering design and analysis. Results of the laboratory testing program along with a brief description and relevant information regarding testing procedures are included in Appendix B.

In addition, GeoTek provided HDR\Schiff with five (5) samples for laboratory testing. HDR\Schiff tested the samples to evaluate whether the soils have deleterious effects on underground utility piping and concrete structures. HDR\Schiff’s report is also included in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

The subject property is situated toward the southern edge of the Transverse Ranges geomorphic province. The Transverse Ranges are an east-west trending series of steep mountain ranges and valleys. The east-west structure of the Transverse Ranges is unique to the normal northwest trend of the adjacent provinces, most closely reflected by the Peninsular Ranges to the south. The Transverse Ranges province extends offshore to the west (San Miguel, Santa Rosa, and Santa Cruz islands), and the San Bernardino Mountains to the east. The San Andreas Fault has displaced the San Bernardino Mountains toward the southeastern
portion of the province. Intense north-south compression is squeezing the Transverse Ranges. As a result, this province is rapidly rising.

More specific to the subject property, the site is located in an area geologically mapped to be underlain by Quaternary age young alluvial fan deposits (see Figure 3; Morton and Miller, 2006). No active faults are known to exist in the immediate vicinity of the subject site. The nearest known active fault to the site is the Cucamonga, located several kilometers to the northeast.

4.2 GENERAL SOIL/GEOLeGIC CONDITIONS

A brief description of the earth materials encountered below the subject lot is presented in the following section. Based on our field exploration and observations, the site is mostly underlain by a relatively thin layer of undocumented fill soils overlying native alluvial deposits. Thicker fill zones are possible.

4.2.1 Fill Soils

Based on our field exploration and site observations, undocumented fill materials underlie portions of the property. These materials were generally observed to consist of silty sandy gravel (see logs in Appendix A). The undocumented fill materials encountered were dry to slightly moist. Maximum depth of the fill encountered in our exploratory trenches was roughly two (2) feet (see Appendix A, Trench T-4), with deeper zones likely. Undocumented fill is likely deepest beneath the existing buildings and along the perimeters of the property associated with the small slopes or improvements (wall/standard) that bound the property. For the remainder of the site, the existing fill appears to be surficial in nature, and likely on the order of a foot or in depth.

4.2.2 Alluvium

Quaternary-age young alluvium (fan deposits) was encountered in all of the trenches excavated on the site, beneath the undocumented fill (if encountered) described above. In general, the alluvial materials typically consist of sandy gravel with cobbles and boulders. According to the results of the laboratory testing performed, the sample of alluvium material tested indicated a “very low” expansion potential (El = 0) when tested and classified in accordance with ASTM D 4829. The test result is shown in Appendix B.
4.3 SURFACE AND GROUNDWATER

4.3.1 Surface Water

If encountered during the earthwork construction, surface water on this site is the result of precipitation or surface run-off from surrounding sites. Overall surface drainage in the area is variable, and most commonly directed toward the nearest street or alley. Provisions for surface drainage will need to be accounted for by the project civil engineer, if necessary.

4.3.2 Groundwater

Groundwater was not encountered in our exploratory excavations. Based on GeoTek’s experience in the area, and other geotechnical reports reviewed for the area, groundwater depth in the immediate site vicinity is greater than 100 feet. However, groundwater or localized seepage can occur due to variations in rainfall, irrigation practices, and other factors not evident at the time of this investigation.

4.4 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within an “Alquist-Priolo” Earthquake Fault Zone (Bryant and Hart, 2007).

4.4.1 Seismic Design Parameters

The site is located at approximately 34.1210 Latitude and -117.7261 Longitude. Site spectral accelerations (Sa and Si), for 0.2 and 1.0 second periods for a Class “D” site, was determined from the USGS Website, Earthquake Hazards Program, Interpolated Probabilistic Ground Motion for the Conterminous 48 States by Latitude/Longitude, 2009 Data. The results are presented in the following table:
### SITE SEISMIC PARAMETERS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mapped 0.2 sec Period Spectral Acceleration, S₂</td>
<td>2.628g</td>
</tr>
<tr>
<td>Mapped 1.0 sec Period Spectral Acceleration, S₁</td>
<td>1.011g</td>
</tr>
<tr>
<td>Site Coefficient for Site Class “D”, Fₐ</td>
<td>1.0</td>
</tr>
<tr>
<td>Site Coefficient for Site Class “D”, Fᵥ</td>
<td>1.5</td>
</tr>
<tr>
<td>Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S₂₅₆ₐ</td>
<td>2.628g</td>
</tr>
<tr>
<td>Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S₂₅₆ₐ</td>
<td>1.516g</td>
</tr>
<tr>
<td>5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S₀₅₆ₐ</td>
<td>1.752g</td>
</tr>
<tr>
<td>5% Damped Design Spectral Response Acceleration Parameter at 1 second, S₀₅₆ₐ</td>
<td>1.011g</td>
</tr>
</tbody>
</table>

#### 4.5 LIQUEFACTION AND SEISMICALLY INDUCED SETTLEMENT

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to lateral movement, sliding, consolidation and settlement of loose sediments, sand boils and other damaging deformations. This phenomenon occurs only below the water table, but, after liquefaction occurs, the liquefied soil/water matrix can propagate upward into overlying non-saturated soil as excess pore water dissipates.

The factors known to influence liquefaction potential include soil type and grain size, relative density, groundwater level, confining pressures, and both intensity and duration of ground shaking. In general, materials that are susceptible to liquefaction are loose, saturated granular soils having low fines content under low confining pressures.

The liquefaction potential on the site is considered negligible due to the nature of the underlying materials (relatively dense older alluvium) and lack of a shallow groundwater table. Additionally, the project site is not in an area mapped by the State of California as a liquefaction hazard zone (CGS, 2000).

#### 4.6 OTHER SEISMIC HAZARDS

Evidence of ancient landslides or slope instabilities at this site was not observed during our investigation. The project site is relatively flat and is not in an area mapped by the State of
California as an earthquake-induced landslide hazard zone (CGS, 2000). Thus, the potential for landslides is considered negligible.

The potential for secondary seismic hazards such as a seiche or tsunami is considered negligible due to site elevation and distance to an open body of water.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Anticipated site development appears feasible from a geotechnical viewpoint provided that the following recommendations are incorporated into the design and construction phases of development. Site development and grading plans should be reviewed by GeoTek when they become available. Cuts and fills on the order of up to five (5) are anticipated for the subject development.

5.2 EARTHWORK CONSIDERATIONS

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the City of Claremont, the 2010 California Building Code (CBC), and recommendations contained in this report. The Grading Guidelines included in Appendix C outline general procedures and do not anticipate all site-specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix C.

5.2.1 Site Clearing and Demolition

In areas of planned grading or improvements, the site should be cleared of existing improvements, vegetation, roots, trash and debris, and properly disposed of offsite. Voids resulting from razing the existing site structures and improvements should be replaced with engineered fill materials with expansion characteristics similar to the onsite materials.

5.2.3 Removals/Overexcavations

If not removed by the proposed grading, all artificial (undocumented) fill materials and the upper one (1) to three (3) feet of alluvium are relatively loose and dry and are subject to complete removal and recompaction within the limits of grading. Depending on actual field conditions encountered during grading, locally deeper areas of removal may be required. The
bottom of all remedial excavations should be scarified to a minimum depth of eight (8) inches, brought to at least optimum moisture content and then recompacted to minimum project standards.

5.2.2 Fills

The onsite soils are considered suitable for reuse as engineered fill provided they are free from vegetation, debris and other deleterious material. The undercut areas should be brought to final subgrade elevations with fill materials that are placed and compacted in general accordance with minimum project standards. Fill materials should be placed at or above optimum moisture content and should be compacted to a minimum relative compaction of 90% as determined by ASTM Test Method D 1557.

Rocks or rock fills that may be generated from the materials excavated onsite should be placed in accordance with the procedures presented in Appendix C (see Pages C-6 through C-8, Limited Larger Rock & Structural Rock Fills). Additionally, rock fragments or cobbles less than six (6) inches in diameter may be utilized in the fill, provided they are not placed in concentrated pockets; there is a sufficient percentage of fine-grained material (i.e., silts and sands) to surround the rocks; and, the distribution of the rocks in the overall fill is observed by and acceptable to our field representative.

Rocks greater than six (6) inches in diameter should not be placed within three (3) feet of finish grades, and should be placed in accordance with the procedures presented in Appendix C. Large rock should also not be placed where utility trenching is anticipated.

5.2.4 Cut and Transition Lots

All cut lots and cut portions of transition lots should be overexcavated a minimum of three (3) feet below finish pad grade or a minimum of two (2) feet below the bottom of the deepest proposed footing, whichever is deeper. Prior to replacing the overexcavated area with very low expansive compacted fill, the exposed removal bottom should be scarified to a minimum depth of eight (8) inches, brought to at least optimum moisture content and then recompacted to minimum project standards. Overexcavations should extend five (5) feet outside the proposed building envelope(s). Overexcavations are recommended to provide a more uniform fill cap and decrease the potential for future differential settlement.

5.2.5 Excavation Characteristics

Excavation in the onsite soil materials is expected to be easy using heavy-duty grading equipment in good operating conditions. All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with local and Cal-
OSHA guidelines. Temporary excavations within the onsite materials should be stable at 1:1 (H:V) inclinations for cuts less than 5 feet in height.

5.2.6 Shrinkage and Subsidence

Several factors will impact earthwork balancing on the site, including shrinkage, bulking, subsidence, trench spoil from utilities and footing excavations, as well as the accuracy of topography.

Shrinkage, bulking and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 0 to 10 percent may be considered for the materials requiring removal and/or recompaction. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of site earthwork construction. Subsidence and bulking are not considered to be factors with the underlying materials within the vicinity of the proposed construction.

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation Design Criteria

Foundation design criteria for a conventional foundation system, in general conformance with the 2010 CBC, are presented herein. Based on the results of our recent laboratory testing, the onsite soils near subgrade may be classified as “very low” expansive soils (0≤EI≤20) in accordance with ASTM D 4829. Below is typical design criteria for the area based on “very low” expansion potential. These minimal recommendations and are not intended to supersede the design by the structural engineer.

The foundation elements for the proposed structures and other improvements should be founded entirely in engineered fill soils. Foundations should be designed in accordance with the 2010 California Building Code (CBC).

Additional testing of the soils should be performed during construction to evaluate the as-graded conditions. Additional recommendations may be necessary based on the as-graded soils conditions.

A summary of our foundation design recommendations is presented below:
MINIMUM DESIGN REQUIREMENTS

<table>
<thead>
<tr>
<th>DESIGN PARAMETER</th>
<th>0≤EI≤20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)</td>
<td>One-Story − 12&lt;br&gt;Two-Story − 18</td>
</tr>
<tr>
<td>Minimum Foundation Width (Inches)*</td>
<td>12*</td>
</tr>
<tr>
<td>Minimum Slab Thickness (inches)</td>
<td>4</td>
</tr>
<tr>
<td>Minimum Slab Reinforcing**</td>
<td>No. 3 rebar&lt;br&gt;24&quot; on-center, each way, placed in the middle 1/3 of the slab**</td>
</tr>
<tr>
<td>Minimum Footing Reinforcement</td>
<td>Two (2) No. 4 Reinforcing Bars:&lt;br&gt;One (1) top and One (1) bottom</td>
</tr>
<tr>
<td>Presaturation of Subgrade Soil (Percent of Optimum/Depth in Inches)</td>
<td>100% to a depth of 12 inches</td>
</tr>
</tbody>
</table>

*Code minimums per Table 1809.7 of the 2010 CBC should be complied with.

**For non-structural residential slabs designed for "very low" expansion potential conditions, GeoTek does not object to the use of 6"x6" − W1.4xW1.4 WWF. In addition to soil expansion characteristics, other geotechnical and/or structural constraints may exist and should be considered into the slab design by the structural engineer as appropriate.

It should be noted that the above recommendations are based on soil support characteristics only. The structural engineer should design the slab and beam reinforcement based on actual loading conditions. If it is desired to utilize post-tensioned foundations, then those recommendations can be provided at the appropriate time.

The following criteria for design of foundations should be implemented into design:

5.3.1.1 An allowable bearing capacity of 1500 pounds per square foot (psf) may be used for design of continuous and perimeter footings 12 inches deep and 12 inches wide, and pad footings 24 inches square and 12 inches deep. This value may be increased by 300 pounds per square foot for each additional 12 inches in depth and 150 pounds per square foot for each additional 12 inches in width to a maximum value of 2500 psf.

The passive earth pressure may be computed as an equivalent fluid having a density of 350 psf per foot of depth, to a maximum earth pressure of 2500 psf for footings founded on engineered fill soils. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. The upper one (1) foot of soil below the adjacent grade should not be used in calculating passive pressure. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.
The above values may be increased as allowed by Code to resist short-term transient loads (e.g. seismic and wind loads).

5.3.1.2 Based on our experience in the area, structural foundations may be designed to withstand a total static settlement of 1 inch and a corresponding maximum differential settlement of one-half of the total settlement over a horizontal distance of 30 feet.

The foundation engineer should incorporate these settlement estimates from the structural loads into the design of the slab, as appropriate.

5.3.1.3 A grade beam, 12 inches wide, should be utilized across large opening or garage entrances. The base of the grade beam should be at the same elevation as the bottom of the adjoining footings (minimum 12 inches in depth).

5.3.1.4 A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2010 California Green Building Standards Code (CALGreen) Section 4.505.2 and the 2010 CBC Section 1910.1.

It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as a result of construction related punctures (e.g. stake penetrations, tears, punctures from walking on the aggregate layer, etc.). These occurrences should be limited as much as possible during construction. Thicker membranes are generally more resistant to accidental puncture than thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant. Although the CBC specifies a 6 mil vapor retarder membrane, it is GeoTek’s opinion that a minimum 10 mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional.

Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and environmental conditions. Ultimately, the vapor retarding system should be comprised of suitable elements to limit migration of water and reduce transmission of water vapor through the slab to acceptable levels. The selected elements should have suitable properties (i.e., thickness, composition, strength, and permeance) to achieve the desired
performance level. Consideration should be given to consulting with an individual possessing specific expertise in this area for additional evaluation.

5.3.2 Miscellaneous Foundation Recommendations

- If used, WWF should be placed on “chairs” or other approved supports and should be located in the middle 1/3 of the slab. “Hooking” of the steel is not an acceptable method of positioning the wire reinforcement in the slab.

- Isolated exterior footings should be tied back to the main foundation system, preferably in two (2) orthogonal directions.

- To reduce moisture penetration beneath the slab on grade areas, utility trenches should be backfilled with engineered fill, lean concrete or concrete slurry where they intercept the perimeter footing or thickened slab edge.

- Soils from the footing excavations should not be placed in the slab-on-grade areas unless properly compacted and tested. The excavations should be free of loose/sloughed materials and be neatly trimmed at the time of concrete placement.

- Under-slab utility trenches should be compacted to project specifications. Compaction should be achieved with a mechanical compaction device. If backfill soils have dried out, they should be thoroughly moisture conditioned prior to placement in trenches.

- Utility trench excavations should be shored or laid back in accordance with applicable CAL/OSHA standards.

- On-site materials may not be suitable for use as bedding material, but will be suitable as backfill provided oversized materials are removed. Jetting of native soils will not be acceptable.

5.3.3 Foundation Set Backs

Minimum setbacks to all foundations should comply with the 2010 CBC or City of Claremont requirements, whichever is greater. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movements and/or differential settlements. If large enough, these movements can compromise the integrity of the improvements. The following recommendations are presented:

- The outside bottom edge of all footings should be set back a minimum of H/3 (where H is the slope height) from the face of any descending slope. The setback should be at least seven (7) feet and need not exceed 40 feet.
The bottom of all footings for structures near retaining walls should be deepened so as to extend below a 1:1 projection upward from the bottom inside edge of the wall stem.

The bottom of any existing foundations for structures should be deepened so as to extend below a 1:1 projection upward from the bottom of the nearest excavation.

### 5.3.4 Retaining and Garden Wall Design and Construction

#### General Design Criteria

Recommendations presented herein apply to typical masonry or concrete vertical retaining walls to a maximum height of up to 10 feet. Additional review and recommendations should be requested for higher walls. These are typical design criteria and are not intended to supersede the design by the structural engineer.

Retaining wall foundations embedded a minimum of 18 inches into engineered fill should be designed using an allowable bearing capacity of 1800 psf. This value may be increased by 300 pounds per square foot for each additional 12 inches in depth and 150 pounds per square foot for each additional 12 inches in width to a maximum value of 2500 psf.

The passive earth pressure may be computed as an equivalent fluid having a density of 350 psf per foot of depth, to a maximum earth pressure of 2500 psf. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. The upper one (1) foot of soil below the adjacent grade should not be used in calculating passive pressure.

The above values may be increased as allowed by Code to resist short-term transient loads (e.g. seismic and wind loads).

Continuous footings should have a minimum reinforcement consisting of two (2) No. 4 reinforcing bars, one (1) top and one (1) bottom. Structural needs may govern and should be evaluated by the structural engineer.

#### Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other
superimposed loading conditions such as traffic, structures, seismic events, or adverse geologic conditions.

<table>
<thead>
<tr>
<th>ACTIVE EARTH PRESSURES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface Slope of Retained Materials (h:v)</td>
</tr>
<tr>
<td>-------------------------</td>
</tr>
<tr>
<td>Level</td>
</tr>
<tr>
<td>2:1</td>
</tr>
</tbody>
</table>

* The design pressures assume the backfill materials have an EI ≤ 20 and an SE>30. Backfill zone includes area between back of wall to plane (1:1, h:v) up from back of wall foundation to ground surface.

Additional lateral forces can be induced on retaining walls during an earthquake. For level backfill and a Site Class “D”, the minimum earthquake-induced force (F_eq) should be 10H^2 (lbs/linear foot of wall). This force can be assumed to act at a distance of 0.6H above the base of the wall, where “H” is the height of the retaining wall measured from the base of the footing (in feet).

Retaining Wall Backfill and Drainage

Retaining wall backfill should be materials comprised of materials with EI ≤ 20 and an SE>30. The wall backfill should also include a minimum one (1) foot wide section of 3/4 to 1-inch clean crushed rock (or approved equivalent). The rock should be placed immediately adjacent to the back of wall and extend up from the back drain to within approximately 12 inches of finish grade. The upper 12 inches should consist of compacted onsite materials. Presence of other materials might necessitate revision to the parameters provided and modification of wall designs. The backfill materials should be placed in lifts no greater than eight (8)-inches in thickness and compacted to a minimum of 90% relative compaction in accordance with ASTM Test Method D 1557. Proper surface drainage needs to be provided and maintained.

Retaining walls should be provided with an adequate pipe and gravel back drain system to prevent build up of hydrostatic pressures. Backdrains should consist of a four (4)-inch diameter perforated collector pipe (Schedule 40, SDR 35, or approved equivalent) embedded in a minimum of one (1) cubic foot per lineal foot of ¾- to 1-inch clean crushed rock or equivalent, wrapped in filter fabric (Mirafi 140N or approved equivalent). The drain system should be connected to a suitable outlet. A minimum of two (2) outlets should be provided for each drain section. Spacing between drain outlets should not exceed 50 feet.
Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

Walls from two (2) to four (4) feet in height may be drained using localized gravel packs behind weep holes at 10 feet maximum spacing (e.g. approximately 1.5 cubic feet of gravel in a woven plastic bag). Weep holes should be provided or the head joints omitted in the first course of block extended above the ground surface. However, nuisance water may still collect in front of the wall.

**Restrained Retaining Walls**

Retaining walls that will be restrained prior to placing and compacting backfill material or that have reentrant or male corners, should be designed for an at-rest equivalent fluid pressure of 65 pcf, plus any applicable surcharge loading. For areas of male or reentrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall laterally from the corner, or a distance otherwise determined by the structural engineer.

**Other Design Considerations**

- Retaining and garden wall foundation elements should be designed in accordance with building code setback requirements. A minimum horizontal setback distance of seven (7) feet as measured from the bottom outside edge of the footing to slope face is recommended.
- Passive earth pressure coefficients used in the design of retaining and garden walls should consider descending slope conditions. Passive pressures should be reduced by one-half in the case of descending 2:1 (h:v) gradient slopes.
- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.
- No backfill should be placed against concrete until minimum design strengths are evidenced by compression tests of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should by approved the project geotechnical engineer or their authorized representative.
- Positive separations should be provided in garden walls at distances not exceeding 20 horizontal feet.
5.3.5 Soil Corrosivity

A soil corrosivity study was performed by HDR|Schiff. A report presenting their findings and recommendations is included in Appendix B.

5.3.6 Soil Sulfate Content

A soil corrosivity study was performed by HDR|Schiff. A report presenting their findings and recommendations is included in Appendix B.

5.3.7 Import Soils

Import soils should have expansion characteristics similar to the onsite soils. GeoTek also recommends that, as a minimum, proposed import soils be tested for soluble sulfate content. GeoTek should be notified a minimum of 72 hours of potential import sources so that appropriate sampling and laboratory testing can be performed.

5.3.8 Concrete Flatwork

5.3.8.1 Exterior Concrete Slabs, Sidewalks and Driveways

Exterior concrete slabs, sidewalks and driveways should be designed using a four (4) inch minimum thickness. No specific reinforcement is required. However, some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices commonly utilized in residential construction.

Sidewalks and driveways may be under the jurisdiction of the governing agency. If so, jurisdictional design and construction criteria would apply, if more restrictive than the recommendations presented herein.

Subgrade soils (typically “very low” expansion potential) should be pre-moistened prior to placing concrete. The subgrade soils below exterior slabs, sidewalks, driveways, etc. at the subject site should be pre-saturated to a minimum of 100% of optimum moisture content to a depth of 12 inches.

All concrete installation, including preparation and compaction of subgrade, should be done in accordance with the City of Claremont and/or County of Los Angeles specifications, and under the observation and testing of GeoTek and a City Inspector, if necessary.

5.3.8.2 Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are essentially unnoticeable to more than 1/8 inch in width. Most cracks in concrete, while unsightly, do not significantly impact long-term performance. While it is possible to take measures (proper
concrete mix, placement, curing, control joints, etc.) to reduce the extent and size of cracks that occur, some cracking will occur despite the best efforts to minimize it. Concrete can also undergo chemical processes that are dependent on a wide range of variables, which are difficult, at best, to control. Concrete, while seemingly a stable material, is also subject to internal expansion and contraction due to external changes over time.

One of the simplest means to control cracking is to provide weakened control joints for cracking to occur along. These do not prevent cracks from developing; they simply provide a relief point for the stresses that develop. These joints are a widely accepted means to control cracks but are not always effective. Control joints are more effective the more closely spaced they are. GeoTek suggests that control joints be placed in two directions and located a distance apart roughly equal to 24 to 36 times the slab thickness.

Exterior concrete flatwork (patios, walkways, driveways, etc.) is often some of the most visible aspects of site development. They are typically given the least level of quality control, being considered “non-structural” components. We suggest that the same standards of care be applied to these features as to the structure itself.

5.4 POST CONSTRUCTION CONSIDERATIONS

5.4.1 Landscape Maintenance and Planting

Water has been shown to weaken the inherent strength of soil, and slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from graded slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Controlling surface drainage and runoff, and maintaining a suitable vegetation cover can minimize erosion. Plants selected for landscaping should be lightweight, deep-rooted types that require little water and are capable of surviving the prevailing climate.

Overwatering should be avoided. The soils should be maintained in a solid to semi-solid state as defined by the materials Atterberg Limits. Care should be taken when adding soil amendments to avoid excessive watering. Leaching as a method of soil preparation prior to planting is not recommended. An abatement program to control ground-burrowing rodents should be implemented and maintained. This is critical as burrowing rodents can decreased the long-term performance of slopes.

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundation. This type of
landscaping should be avoided. If used, then extreme care should be exercised with regard to the irrigation and drainage in these areas.

5.4.2 Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground. Pad drainage should be directed toward approved area(s) and not be blocked by other improvements.

It is the owner's responsibility to maintain and clean drainage devices on or contiguous to their lot. In order to be effective, maintenance should be conducted on a regular and routine schedule and necessary corrections made prior to each rainy season.

5.5 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

We recommend that site grading, specifications, pool, and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. We also recommend that GeoTek representatives be present during site grading and foundation construction to check for proper implementation of the geotechnical recommendations. The owner/developer should verify that GeoTek representatives perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of unsuitable materials.
- Observe and test bottom of removals prior to fill placement.
- Evaluate the suitability of onsite and import materials for fill placement, and collect soil samples for laboratory testing where necessary.
- Observe the fill for uniformity during placement, including utility trenches.
- Perform field density testing of the fill materials.
- Observe and probe foundation excavations to confirm suitability of bearing materials.

If requested, a construction observation and compaction report can be provided by GeoTek, which can comply with the requirements of the governmental agencies having jurisdiction over the project. We recommend that these agencies be notified prior to commencement of construction so that necessary grading permits can be obtained.
6. INTENT

It is the intent of this report to aid in the design and construction of the proposed development. Implementation of the advice presented in Section 5 of this report is intended to reduce risk associated with construction projects. The professional opinions and geotechnical advice contained in this report are not intended to imply total performance of the project or guarantee that unusual or variable conditions will not be discovered during or after construction.

The scope of our evaluation is limited to the boundaries of the subject residential lot. This review does not and should in no way be construed to encompass any areas beyond the specific area of the proposed construction as indicated to us by the client. Further, no evaluation of any existing site improvements is included. The scope is based on our understanding of the project and the client's needs, our fee estimate (P3-0500213) dated April 28, 2013 and geotechnical engineering standards normally used on similar projects in this region.

7. LIMITATIONS

The materials observed on the project site appear to be representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during site construction. Site conditions may vary due to seasonal changes or other factors. GeoTek, Inc. assumes no responsibility or liability for work, testing or recommendations performed or provided by others.

Since our recommendations are based on the site conditions observed and encountered, and laboratory testing, our conclusion and recommendations are professional opinions that are limited to the extent of the available data. Observations during construction are important to allow for any change in recommendations found to be warranted. These opinions have been derived in accordance with current standards of practice and no warranty is expressed or implied. Standards of practice are subject to change with time.
8. SELECTED REFERENCES


California Department of Water Resources groundwater well data (http://wdl.water.ca.gov).


GeoTek, Inc., In-house proprietary information.


Figure 2
Exploration Location Map
APPENDIX A

LOGS OF EXPLORATORY TRENCHES

Serrano II Project
City of Claremont, Los Angeles County, California
Project No. 1034-CR3
A - FIELD TESTING AND SAMPLING PROCEDURES

Bulk Samples (Large)
These samples are normally large bags of earth materials over 20 pounds in weight collected from the field by means of hand digging or exploratory cuttings.

Bulk Samples (Small)
These are plastic bag samples which are normally airtight and contain less than 5 pounds in weight of earth materials collected from the field by means of hand digging or exploratory cuttings. These samples are primarily used for determining natural moisture content and classification indices.

B - TRENCH LOG LEGEND
The following abbreviations and symbols often appear in the classification and description of soil and rock on the logs of trenches:

SOILS
USCS      Unified Soil Classification System
f-c       Fine to coarse
f-m       Fine to medium

GEOLOGIC
B: Attitudes  Bedding: strike/dip
J: Attitudes  Joint: strike/dip
C: Contact line
        Dashed line denotes USCS material change
        Solid Line denotes unit / formational change
        Thick solid line denotes end of trench

(Additional denotations and symbols are provided on the log of trenches)
**GeoTek, Inc.**
**LOG OF EXPLORATORY TRENCH**

**PROJECT NO.:** 1074-CR3  
**LOGGED BY:** IMP  
**PROJECT NAME:** Serrano II - Claremont  
**EQUIPMENT:** Backhoe  
**CLIENT:** D.R. Horton  
**DATE:** 7/24/2013  
**LOCATION:** See Exploration Location Map

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample Type</th>
<th>Sample Number</th>
<th>USGS Symbol</th>
<th>Material Description and Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>GW</td>
<td>BI</td>
<td>Alluvium</td>
<td>0'-4': Sandy GRAVEL with cobbles and boulders, gray brown, dry to slightly moist, f-c sand, f-c gravels, abundant rounded cobbles.</td>
</tr>
</tbody>
</table>

**TRENCH TERMINATED AT 4 FEET**

No Groundwater Encountered  
Backfilled with Trench Spoils

---

**Legend**
- **Sample Types:**
  - Ring Sample
  - Large Bulk Sample
  - Water Table

**Laboratory Test:**
- AL = Atterberg Limits  
- EI = Expansion Index  
- MD = Maximum Density  
- SA = Sieve Analysis  
- SR = Sulfate/Resistivity Test  
- SH = Shear Testing  
- RV = R-Value Test  
- CO = Consolidation

---

D-30
# GeoTek, Inc.
## LOG OF EXPLORATORY TRENCH

**PROJECT NO.:** 1034-CR3  
**LOGGED BY:**  
**PROJECT NAME:** Serrano II - Claremont  
**EQUIPMENT:** Backhoe  
**CLIENT:** D.R. Horton  
**LOCATION:** See Exploration Location Map  
**DATE:** 7/24/2013

### TRENCH NO.: T-2

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>SAMPLES</th>
<th>LCSC Symbol</th>
<th>Field Testing</th>
<th>Laboratory Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 4.5</td>
<td>BI</td>
<td>GW</td>
<td>Alluvium</td>
<td>Water Content (%)</td>
</tr>
<tr>
<td>0 - 4.5</td>
<td>BI</td>
<td>GW</td>
<td>Alluvium</td>
<td>0.5</td>
</tr>
</tbody>
</table>

**MATERIAL DESCRIPTION AND COMMENTS**

- BI: Sandy GRAVEL with cobbles and boulders, gray brown, dry to slightly moist, f-c sand, f-c gravels, abundant rounded cobbles.

**TRENCH TERMINATED AT 4.5 FEET**

- No Groundwater Encountered
- Backfilled with Trench Spoils

**LEGEND**

- **Sample Type:**
  - Ring Sample
  - Large Bulk Sample
  - Water Table

- **Laboratory Testing:**
  - AL = Atterberg Limits
  - BI = Expansion Index
  - MD = Maximum Density
  - SA = Sieve Analysis
  - SR = Sulfate/Resistance Test
  - SH = Shear Testing
  - RV = R-Value Test
  - CO = Consolidaion
**GeoTek, Inc.**
**LOG OF EXPLORATORY TRENCH**

**PROJECT NO.:** 1034-CR3
**PROJECT NAME:** Serrano II - Claremont
**CLIENT:** D.R. Horton
**LOCATION:** See Exploration Location Map

**LOGGED BY:** JMP
**EQUIPMENT:** Backhoe
**DATE:** 7/24/2013

---

### TRENCH NO.: T-3

**MATERIAL DESCRIPTION AND COMMENTS**

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<tr>
<th>Depth (ft)</th>
<th>Sample Type</th>
<th>Sample Number</th>
<th>USCS Symbol</th>
<th>Field Testing</th>
<th>Laboratory Testing</th>
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<td>0.0-1.5</td>
<td>GW</td>
<td>BI</td>
<td>Undocumented Artificial Fill</td>
<td></td>
<td>SR</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.0-1.5: Silty sandy GRAVEL, brown, dry to slightly moist, f-m sand, f-c gravels, some rounded cobbles, roots.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5-4.5</td>
<td>GW</td>
<td></td>
<td>Alluvium</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.5-4.5: Sandy GRAVEL with cobbles, gray brown, slightly moist, f-c sand, f-c gravels, abundant rounded cobbles, few rounded boulders.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**TRENCH TERMINATED AT 4.5 FEET**

No Groundwater Encountered
Backfilled with Trench Spoils

---

**LEGEND**

- **Sample Type:**
  - Ring Sample
  - Large Bulk Sample
  - Water Table

- **Laboratory Testing:**
  - AL = Atterberg Limits
  - EI = Expansion Index
  - MD = Maximum Density
  - SA = Sieve Analysis
  - SR = Sulfate/Resistivity Test
  - SH = Shear Testing
  - RV = R-Value Test
  - CO = Consolidation
# GeoTek, Inc.
## LOG OF EXPLORATORY TRENCH

**PROJECT NO.:** 1034-CR3  
**LOGGED BY:** JMP  
**PROJECT NAME:** Serrano II - Claremont  
**EQUIPMENT:** Backhoe  
**CLIENT:** D.R. Horton  
**LOCATION:** See Exploration Location Map  
**DATE:** 7/24/2013

### TRENCH NO.: T-4

#### MATERIAL DESCRIPTION AND COMMENTS

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<th>Sample Type</th>
<th>Sample Number</th>
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<th>Field Testing</th>
<th>Laboratory Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BI</td>
<td>GW</td>
<td>Undocumented Artificial Fill</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0'-2'</td>
<td>Silty sandy GRAVEL, brown, dry, f-m sand, f-c gravels, some rounded cobbles and boulders.</td>
<td></td>
<td>SR</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>GW</td>
<td>Alluvium</td>
<td>2'-4': Silty Sandy GRAVEL with cobbles, gray brown, dry, f-m sand, f-c gravels, abundant rounded cobbles, some rounded boulders.</td>
<td></td>
<td></td>
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</table>

**TRENCH TERMINATED AT 4 FEET**

No Groundwater Encountered  
Backfilled with Trench Spoils

---

**LEGEND**

- Sample Type:  
  - **R** = Ring Sample  
  - **L** = Large Bulk Sample

**Laboratory Testing:**

- **AL** = Atterberg Limits  
  - **BI** = Expansion Index  
  - **MD** = Maximum Density  
  - **SA** = Sieve Analysis  
  - **SR** = Sulfate/Residuary Tests  
  - **SH** = Shear Testing  
  - **RV** = R-Value Test  
  - **CO** = Consolidation

---

D-33
**Log of Exploratory Trench**

**Trench No.: T-5**

**Material Description and Comments**

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<th>Sample Type</th>
<th>Sample Number</th>
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<th>Field Testing</th>
<th>Laboratory Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>GW</td>
<td>0.5-4.5: Sandy GRAVEL with cobbles and boulders, gray brown, dry, f-c sand, f-c gravels, abundant rounded cobbles.</td>
<td>AL, E, L, S, MD, SR</td>
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</tbody>
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**Trench Terminated at 4.5 Feet**

No Groundwater Encountered
Backfilled with Trench Spoils

---

**Legend**
- Ring Sample
- Large Bulk Sample
- Water Table

**Laboratory Testing:**
- AL = Atterberg Limits
- BI = Expansion Index
- MD = Maximum Density
- SA = Sieve Analysis
- SR = Sulfate/Resistivity Test
- SH = Shrinkage Test
- RV = R-Value Test
- CO = Consolidation
APPENDIX B

LABORATORY TESTING RESULTS

Serrano II Project
City of Claremont, Los Angeles County, California
Project No. 1034-CR3
SUMMARY OF LABORATORY TESTING

Classification
Soils were classified visually in general accordance to the Unified Soil Classification System (ASTM Test Method D 2487). The soil classifications are shown on the log of trenches in Appendix A.

Moisture-Density Relationship
Laboratory testing was performed on a sample collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for the soil type was determined in general accordance with test method ASTM Test Procedure D 1557. The results are included herein.

Expansion Index
Expansion Index testing was performed on a soil sample. Testing was performed in general accordance with ASTM Test Method D 4829. The result of the testing is included herein.

Sulfate Content
Analysis to determine the water-soluble sulfate content was performed by others. The results of the testing are included herein.

Chloride Content
Analysis to determine the water-soluble chloride content was performed by others. The results of the testing are included herein.

Resistivity and pH
Analysis to determine the resistivity and pH was performed by others. The results of the testing are included herein.

Direct Shear
Shear testing was performed in a direct shear machine of the strain-control type in general accordance with ASTM Test Method D 3080. The rate of deformation is approximately 0.025 inches per minute. The samples were sheared under varying confining loads in order to determine the coulomb shear strength parameters, angle of internal friction and cohesion. One test was performed on a bulk sample that was remolded to 90 percent relative compaction. The results of the testing are included herein.
MOISTURE/DENSITY RELATIONSHIP

Client: DR Horton
Project: Serrano II
Location: Claremont
Material Type: Gravelly Silty Sand
Material Supplier: N/A
Material Source: N/A
Sample Location: T-5 @ 0-4.5

Sampled By: JMP
Received By: DI
Tested By: DI
Reviewed By: JMP

Test Procedure: ASTM 1557

Method: C

Oversized Material (%): 26.6
Correction Required: yes

DRY DENSITY (pcf):
CORRECTED DRY DENSITY (pcf):
ZERO AIR VOIDS DRY DENSITY (pcf):
S.G. 2.7
S.G. 2.8
S.G. 2.6
OVERSIZE CORRECTED
ZERO AIR VOIDS
Poly. (DRY DENSITY (pcf):
Poly. (S.G. 2.7)
Poly. (S.G. 2.8)
Poly. (S.G. 2.6)

MOISTURE DENSITY RELATIONSHIP VALUES
Maximum Dry Density, pcf 131.5
Corrected Maximum Dry Density, pcf

@ Optimum Moisture, % 7.5

Grain Size Distribution:
% Gravel (retained on No. 4)
% Sand (Passing No. 4, Retained on No. 200)
% Silt and Clay (Passing No. 200)
Classification:
Unified Soils Classification:
AASHTO Soils Classification:

Atterberg Limits:
Liquid Limit, %
Plastic Limit, %
Plasticity Index, %
EXPANSION INDEX TEST
(ASTM D4829)

Client: DR Horton
Project Number: 1034-CR3
Project Location: Serrano II Claremont

Tested/Checked By: DI Lab No Corona
Date Tested: 8/1/2013
Sample Source: T-5 @ 0-4.5'
Sample Description: 

Ring #: Ring Dia.: 4.01" Ring Ht.: 1"
Loading weight: 5516 grams

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DENSITY DETERMINATION

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READINGS

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SATURATION DETERMINATION

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<td>I</td>
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FINAL MOISTURE

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<thead>
<tr>
<th>Weight of wet sample &amp; tare</th>
<th>Weight of dry sample &amp; tare</th>
<th>Tare</th>
<th>% Moisture</th>
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<tr>
<td>515.0</td>
<td>465.1</td>
<td>80.0</td>
<td>12.7%</td>
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EXPANSION INDEX = 0
DIRECT SHEAR TEST

Project Name: Serrano II
Project Number: 1034-CR3
Soil Description: Gray Brown Gravelly Silty Sand

Sample Source: T-5 @ 0-4.5'
Date Tested: 8/2/2013

Shear Strength: \( \theta = 33.1^\circ \), \( c = 0.00 \text{ ksf} \)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Load (kN)</th>
<th>Final Water Content (%)</th>
<th>Final Dry Density (kbf/sq ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.38</td>
<td>15.0</td>
<td>112.2</td>
</tr>
<tr>
<td>2</td>
<td>2.77</td>
<td>13.6</td>
<td>113.3</td>
</tr>
<tr>
<td>3</td>
<td>5.44</td>
<td>13.6</td>
<td>114.2</td>
</tr>
</tbody>
</table>

Notes:
1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% from a bulk sample collected during the field investigation.
2 - The above reflect residual shear strength at saturated conditions.
3 - The tests were run at a shear rate of 0.025 in/min.
August 7, 2013  

via email:  
jpflueger@geotekusa.com

GEOTEK, INC.  
710 East Parkridge Avenue, Suite 105  
Corona, CA  92879

Attention:  Mr. Jeff Pflueger, PG, CEG

Re: Soil Corrosivity Study  
Serrano II-DR Horton  
Claremont, California  
HDR #216634, GTK #1034-CR3

INTRODUCTION

Laboratory tests have been completed on five soil samples provided for the referenced project. The purpose of these tests was to determine if the soils might have deleterious effects on underground utility piping and concrete structures. HDR Engineering, Inc. (HDR|Schiff) assumes that the samples provided are representative of the most corrosive soils at the site.

The proposed construction consists of single-family residences. The site is located near the intersection of Baseline Road and Mountain Avenue in Claremont, California. The water table depth is reportedly deep enough to not be a concern from a corrosion standpoint.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. Our recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR|Schiff will be happy to work with them as a separate phase of this project.

LABORATORY SOIL CORROSIVITY TESTS

The electrical resistivity of each sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per CTM 643. A 5:1 water:soluble extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327 and D6919. Laboratory analysis was performed under HDR|Schiff number 13-0603SCS and the test results are shown in Table 1.
SOIL CORROSION

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:¹

<table>
<thead>
<tr>
<th>Soil Resistivity in ohm-centimeters</th>
<th>Corrosivity Category</th>
</tr>
</thead>
<tbody>
<tr>
<td>Greater than 10,000</td>
<td>Mildly Corrosive</td>
</tr>
<tr>
<td>2,000 to 10,000</td>
<td>Moderately Corrosive</td>
</tr>
<tr>
<td>1,000 to 2,000</td>
<td>Corrosive</td>
</tr>
<tr>
<td>0 to 1,000</td>
<td>Severely Corrosive</td>
</tr>
</tbody>
</table>

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivities were in the mildly corrosive category with as-received moisture. When saturated, the resistivities were in the mildly to moderately corrosive categories. The resistivities dropped considerably with added moisture because the samples were dry as-received.

Soil pH values varied from 7.1 to 7.7. This range is neutral to mildly alkaline.² These values do not particularly increase soil corrosivity.

The soluble salt content of the samples was low.

Ammonium and nitrate were detected in low concentrations.

Tests were not made for sulfide and negative oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as moderately corrosive to ferrous metals.

CORROSION CONTROL RECOMMENDATIONS

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

Implement all the following measures:

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and possible future cathodic protection.

2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the possible future application of cathodic protection:
   a. At each end of the pipeline.
   b. At each end of all casings.
   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

3. To prevent dissimilar metal corrosion cells and to facilitate the possible future application of cathodic protection, electrically isolate each buried steel pipeline per NACE Standard SP0286 from:
   a. Dissimilar metals.
   b. Dissimilarly coated piping (cement-mortar vs. dielectric).
   c. Above ground steel pipe.
   d. All existing piping.

4. Choose one of the following corrosion control options:

**OPTION 1**

a. Apply a suitable dielectric coating intended for underground use such as:
   i. Polyurethane per AWWA C222 or
   ii. Extruded polyethylene per AWWA C215 or
   iii. A tape coating system per AWWA C214 or
   iv. Hot applied coal tar enamel per AWWA C203 or
   v. Fusion bonded epoxy per AWWA C213.

b. Although it is customary to cathodically protect bonded dielectrically coated structures, cathodic protection is not recommended at this time due to moderately...
corrosive soils. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.

OPTION 2

a. As an alternative to dielectric coating and possible future cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete 3 inches thick, using any type of cement. Joint bonds, test stations, and insulated joints are still required for these alternatives.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Iron Pipe

Implement all the following measures:

1. Electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE Standard SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and possible future cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of possible future cathodic protection:
   a. At each end of the pipeline.
   b. At each end of any casings.
   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

OPTION 1

a. Apply a suitable coating intended for underground use such as:
   i. Polyethylene encasement per AWWA C105; or
   ii. Epoxy coating; or
   iii. Polyurethane; or
   iv. Wax tape.

   NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

b. Although it is customary to cathodically protect coated structures, cathodic protection is not recommended at this time due to moderately corrosive soils. Joint bonds, test stations, and insulated joints should still be installed and will facilitate the application of cathodic protection in the future if needed to control leaks.
OPTION 2

a. As an alternative to coating systems described in Option 1 and possible future cathodic protection, concrete encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of cement.

Copper Tubing
Implement all the following measures:

1. Place cold water copper tubing in an 8-mil polyethylene sleeve or encase in double 4-mil thick polyethylene sleeves and bed and backfill with clean sand at least 2 inches thick surrounding the tubing. Clean sand should have a minimum resistivity of no less than 3000 ohm-cm, and a pH of 6.0–8.0. Copper tubing for cold water can also be treated the same as for hot water.

2. Hot water tubing may be subject to a higher corrosion rate. Protect hot copper tubing by one of the following measures:
   a. Preventing soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing with PVC pipe with solvent-welded joints. or
   b. Applying cathodic protection per NACE Standard SP0169. The amount of cathodic protection current needed can be minimized by coating the tubing.

Plastic and Vitrified Clay Pipe

1. No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint.

2. Protect all metallic fittings and valves with wax tape per AWWA C217 or epoxy.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.
Concrete

1. From a corrosion standpoint, any type of cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.1 percent.\textsuperscript{3,4,5}

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentration\textsuperscript{6} found onsite.

CLOSURE

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.

Leonardo Solis

Brien L. Clark, P.E.

Enc: Table 1

\textsuperscript{3} 2009 International Building Code (IBC) which refers to American Concrete Institute (ACI-318) Table 4.3.1
\textsuperscript{4} 2009 International Residential Code (IRC) which refers to American Concrete Institute (ACI-318) Table 4.3.1
\textsuperscript{5} 2010 California Building Code (CBC) which refers to American Concrete Institute (ACI-318) Table 4.3.1
\textsuperscript{6} Design Manual 303: Concrete Cylinder Pipe. Aneron. p.65
### Table 1 - Laboratory Tests on Soil Samples

**GeoTek, Inc.**

**Serrano II-D.R. Horton**

*Your #1031-CR3, HDR/Schiff #13-0603SCS*

*25-Jul-13*

<table>
<thead>
<tr>
<th>Sample ID</th>
<th>T-1 @ 0-4</th>
<th>B1</th>
<th>T-2 @ 0-4-5</th>
<th>B1</th>
<th>T-3 @ 0-4-5</th>
<th>B1</th>
<th>T-4 @ 0-4</th>
<th>B1</th>
<th>T-5 @ 0-4-5</th>
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<td></td>
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<tr>
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<tr>
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</tr>
<tr>
<td><strong>pH</strong></td>
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<td>7.6</td>
<td>7.4</td>
<td>7.1</td>
<td>7.1</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td><strong>Electrical Conductivity</strong></td>
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<td>0.02</td>
<td>0.10</td>
<td>0.06</td>
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<td></td>
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</tr>
</tbody>
</table>

#### Chemical Analyses

**Cations**

- calcium $\text{Ca}^{2+}$ mg/kg: 31, 22, 55, 38, 40
- magnesium $\text{Mg}^{2+}$ mg/kg: 4.6, 3.7, 7.1, 6.5, 4.5
- sodium $\text{Na}^+$ mg/kg: 7.0, 4.6, 27, 17, 10
- potassium $\text{K}^+$ mg/kg: 6.4, 7.6, 15, 17, 13

**Anions**

- carbonate $\text{CO}_3^{2-}$ mg/kg: ND, ND, ND, ND, ND
- bicarbonate $\text{HCO}_3^-$ mg/kg: 70, 46, 92, 88, 82
- fluoride $\text{F}^-$ mg/kg: 1.0, 0.8, 1.2, 2.2, 1.4
- chloride $\text{Cl}^-$ mg/kg: 0.9, 1.0, 6.7, 4.9, 5.9
- sulfate $\text{SO}_4^{2-}$ mg/kg: 2.6, 2.7, 83, 8.9, 11
- phosphate $\text{PO}_4^{3-}$ mg/kg: 0.9, 10, 23, 47, 22

**Other Tests**

- ammonium $\text{NH}_4^+$ mg/kg: 0.7, 0.6, 1.4, 1.2, 0.6
- nitrate $\text{NO}_3^-$ mg/kg: 3.4, 11, 26, 22, 5.1
- sulfide $\text{S}^-$ qual: na, na, na, na, na
- Redox mV: na, na, na, na, na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract. mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed
APPENDIX C

GENERAL GRADING GUIDELINES

Serrano II Project
City of Claremont, Los Angeles County, California
Project No. 1034-CR3
GENERAL GRADING GUIDELINES

Guidelines presented herein are intended to address general construction procedures for earthwork construction. Specific situations and conditions often arise which cannot reasonably be discussed in general guidelines, when anticipated these are discussed in the text of the report. Often unanticipated conditions are encountered which may necessitate modification or changes to these guidelines. It is our hope that these will assist the contractor to more efficiently complete the project by providing a reasonable understanding of the procedures that would be expected during earthwork and the testing and observation used to evaluate those procedures.

General
Grading should be performed to at least the minimum requirements of governing agencies, Chapters 18 and 33 of the Uniform Building Code, CBC (2010) and the guidelines presented below.

Preconstruction Meeting
A preconstruction meeting should be held prior to site earthwork. Any questions the contractor has regarding our recommendations, general site conditions, apparent discrepancies between reported and actual conditions and/or differences in procedures the contractor intends to use should be brought up at that meeting. The contractor (including the main onsite representative) should review our report and these guidelines in advance of the meeting. Any comments the contractor may have regarding these guidelines should be brought up at that meeting.

Grading Observation and Testing

1. Observation of the fill placement should be provided by our representative during grading. Verbal communication during the course of each day will be used to inform the contractor of test results. The contractor should receive a copy of the "Daily Field Report" indicating results of field density tests that day. If our representative does not provide the contractor with these reports, our office should be notified.

2. Testing and observation procedures are, by their nature, specific to the work or area observed and location of the tests taken, variability may occur in other locations. The contractor is responsible for the uniformity of the grading operations; our observations and test results are intended to evaluate the contractor’s overall level of efforts during grading. The contractor’s personnel are the only individuals participating in all aspect of site work. Compaction testing and observation should not be considered as relieving the contractor’s responsibility to properly compact the fill.

3. Cleanouts, processed ground to receive fill, key excavations, and subdrains should be observed by our representative prior to placing any fill. It will be the contractor’s responsibility to notify our representative or office when such areas are ready for observation.

4. Density tests may be made on the surface material to receive fill, as considered warranted by this firm.
5. In general, density tests would be made at maximum intervals of two feet of fill height or every 1,000 cubic yards of fill placed. Criteria will vary depending on soil conditions and size of the fill. More frequent testing may be performed. In any case, an adequate number of field density tests should be made to evaluate the required compaction and moisture content is generally being obtained.

6. Laboratory testing to support field test procedures will be performed, as considered warranted, based on conditions encountered (e.g. change of material sources, types, etc.) Every effort will be made to process samples in the laboratory as quickly as possible and in progress construction projects are our first priority. However, laboratory workloads may cause delays and some soils may require a minimum of 48 to 72 hours to complete test procedures. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.

7. Procedures for testing of fill slopes are as follows:
   a) Density tests should be taken periodically during grading on the flat surface of the fill, three to five feet horizontally from the face of the slope.
   b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.

8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

1. All vegetation, and other deleterious materials, should be removed from the site. If material is not immediately removed from the site it should be stockpiled in a designated area(s) well outside of all current work areas and delineated with flagging or other means. Site clearing should be performed in advance of any grading in a specific area.

2. Efforts should be made by the contractor to remove all organic or other deleterious material from the fill, as even the most diligent efforts may result in the incorporation of some materials. This is especially important when grading is occurring near the natural grade. All equipment operators should be aware of these efforts. Laborers may be required as root pickers.

3. Nonorganic debris or concrete may be placed in deeper fill areas provided the procedures used are observed and found acceptable by our representative. Typical procedures are similar to those indicated on Plate G-4.

Treatment of Existing Ground

1. Following site clearing, all surficial deposits of alluvium and colluvium as well as weathered or creep effected bedrock, should be removed (see Plates G-1, G-2 and G-3) unless otherwise specifically indicated in the text of this report.
2. In some cases, removal may be recommended to a specified depth (e.g. flat sites where partial alluvial removals may be sufficient). The contractor should not exceed these depths unless directed otherwise by our representative.

3. Groundwater existing in alluvial areas may make excavation difficult. Deeper removals than indicated in the text of the report may be necessary due to saturation during winter months.

4. Subsequent to removals, the natural ground should be processed to a depth of six inches, moistened to near optimum moisture conditions and compacted to fill standards.

5. Exploratory backhoe or dozer trenches still remaining after site removal should be excavated and filled with compacted fill if they can be located.

Subdrainage

1. Subdrainage systems should be provided in canyon bottoms prior to placing fill, and behind buttress and stabilization fills and in other areas indicated in the report. Subdrains should conform to schematic diagrams G-1 and G-5, and be acceptable to our representative.

2. For canyon subdrains, runs less than 500 feet may use six-inch pipe. Typically, runs in excess of 500 feet should have the lower end as eight-inch minimum.

3. Filter material should be clean, 1/2 to 1-inch gravel wrapped in a suitable filter fabric. Class 2 permeable filter material per California Department of Transportation Standards tested by this office to verify its suitability, may be used without filter fabric. A sample of the material should be provided to the Soils Engineer by the contractor at least two working days before it is delivered to the site. The filter should be clean with a wide range of sizes.

4. Approximate delineation of anticipated subdrain locations may be offered at 40-scale plan review stage. During grading, this office would evaluate the necessity of placing additional drains.

5. All subdrainage systems should be observed by our representative during construction and prior to covering with compacted fill.

6. Subdrains should outlet into storm drains where possible. Outlets should be located and protected. The need for backflow preventers should be assessed during construction.

7. Consideration should be given to having subdrains located by the project surveyors.

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).

2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.

3. If the moisture content or relative density varies from that recommended by this firm, the contractor should rework the fill until it is in accordance with the following:
a) Moisture content of the fill should be at or above optimum moisture. Moisture should be evenly distributed without wet and dry pockets. Pre-watering of cut or removal areas should be considered in addition to watering during fill placement, particularly in clay or dry surficial soils. The ability of the contractor to obtain the proper moisture content will control production rates.

b) Each six-inch layer should be compacted to at least 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency. In most cases, the testing method is ASTM Test Designation D 1557.

4. Rock fragments less than eight inches in diameter may be utilized in the fill, provided:

a) They are not placed in concentrated pockets;

b) There is a sufficient percentage of fine-grained material to surround the rocks;

c) The distribution of the rocks is observed by, and acceptable to, our representative.

5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated suitable for rock disposal (see Plate G-4). On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If significant oversize materials are encountered during construction, these guidelines should be requested.

6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepfoot compactors or other suitable methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

1. The contractor should obtain a minimum relative compaction of 90 percent out to the finished slope face of fill slopes. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment.

2. Slopes trimmed to the compacted core should be overbuilt by at least three (3) feet with compaction efforts out to the edge of the false slope. Failure to properly compact the outer edge results in trimming not exposing the compacted core and additional compaction after trimming may be necessary.

3. If fill slopes are built "at grade" using direct compaction methods, then the slope construction should be performed so that a constant gradient is maintained throughout construction. Soil should not be "spilled" over the slope face nor should slopes be "pushed out" to obtain grades. Compaction equipment should compact each lift along the immediate top of slope. Slopes should be back rolled or otherwise compacted at approximately every 4 feet vertically as the slope is built.

4. Corners and bends in slopes should have special attention during construction as these are the most difficult areas to obtain proper compaction.
5. Cut slopes should be cut to the finished surface. Excessive undercutting and smoothing of the face with fill may necessitate stabilization.

**Keyways, Buttress and Stabilization Fills**

Keyways are needed to provide support for fill slope and various corrective procedures.

1. Side-hill fills should have an equipment-width key at their toe excavated through all surficial soil and into competent material and tilted back into the hill (Plates G-2, G-3). As the fill is elevated, it should be bench through surficial soil and slopewash, and into competent bedrock or other material deemed suitable by our representatives (See Plates G-1, G-2, and G-3).

2. Fill over cut slopes should be constructed in the following manner:
   a) All surficial soils and weathered rock materials should be removed at the cut-fill interface.
   b) A key at least one and one-half (1.5) equipment width wide (or as needed for compaction), and tipped at least one (1) foot into slope, should be excavated into competent materials and observed by our representative.
   c) The cut portion of the slope should be excavated prior to fill placement to evaluate if stabilization is necessary. The contractor should be responsible for any additional earthwork created by placing fill prior to cut excavation. (see Plate G-3 for schematic details.)

3. Daylight cut lots above descending natural slopes may require removal and replacement of the outer portion of the lot. A schematic diagram for this condition is presented on Plate G-2.

4. A basal key is needed for fill slopes extending over natural slopes. A schematic diagram for this condition is presented on Plate G-2.

5. All fill slopes should be provided with a key unless within the body of a larger overall fill mass. Please refer to Plate G-3 for specific guidelines.

Anticipated buttress and stabilization fills are discussed in the text of the report. The need to stabilize other proposed cut slopes will be evaluated during construction. Plate G-5 shows a schematic of buttress construction.

1. All backcuts should be excavated at gradients of 1:1 or flatter. The backcut configuration should be determined based on the design, exposed conditions, and need to maintain a minimum fill width and provide working room for the equipment.

2. On longer slopes, backcuts and keyways should be excavated in maximum 250 feet long segments. The specific configurations will be determined during construction.

3. All keys should be a minimum of two (2) feet deep at the toe and slope toward the heel at least one foot or two (2%) percent, whichever is greater.

4. Subdrains are to be placed for all stabilization slopes exceeding 10 feet in height. Lower slopes are subject to review. Drains may be required. Guidelines for subdrains are presented on Plate G-5.
5. Benching of backcuts during fill placement is required.

Lot Capping

1. When practical, the upper three (3) feet of material placed below finish grade should be comprised of the least expansive material available. Preferably, highly and very highly expansive materials should not be used. We will attempt to offer advice based on visual evaluations of the materials during grading, but it must be realized that laboratory testing is needed to evaluate the expansive potential of soil. Minimally, this testing takes two (2) to four (4) days to complete.

2. Transition lots (cut and fill) both per plan and those created by remedial grading (e.g., lots above stabilization fills, along daylight lines, above natural slopes, etc.) should be capped with a minimum three foot thick compacted fill blanket.

3. Cut pads should be observed by our representative(s) to evaluate the need for overexcavation and replacement with fill. This may be necessary to reduce water infiltration into highly fractured bedrock or other permeable zones, and/or due to differing expansive potential of materials beneath a structure. The overexcavation should be at least three feet. Deeper overexcavation may be recommended in some cases.

**ROCK PLACEMENT AND ROCK FILL GUIDELINES**

It is anticipated that large quantities of oversize material would be generated during grading. It’s likely that such materials may require special handling for burial. Although alternatives may be developed in the field, the following methods of rock disposal are recommended on a preliminary basis.

**Limited Larger Rock**

When materials encountered are principally soil with limited quantities of larger rock fragments or boulders, placement in windrows is recommended. The following procedures should be applied:

1. Oversize rock (greater than 8 inches) should be placed in windrows.
   a) Windrows are rows of single file rocks placed to avoid nesting or clusters of rock.
   b) Each adjacent rock should be approximately the same size (within ~one foot in diameter).
   c) The maximum rock size allowed in windrows is four feet.

2. A minimum vertical distance of three feet between lifts should be maintained. Also, the windrows should be offset from lift to lift. Rock windrows should not be closer than 15 feet to the face of fill slopes and sufficient space must be maintained for proper slope construction (see Plate G-4).

3. Rocks greater than eight inches in diameter should not be placed within seven feet of the finished subgrade for a roadway or pads and should be held below the depth of the lowest utility. This will allow easier trenching for utility lines.
4. Rocks greater than four feet in diameter should be broken down, if possible, or they may be placed in a dozer trench. Each trench should be excavated into the compacted fill a minimum of one foot deeper than the largest diameter of rock.
   a) The rock should be placed in the trench and granular fill materials (SE>30) should be flooded into the trench to fill voids around the rock.
   b) The over size rock trenches should be no closer together than 15 feet from any slope face.
   c) Trenches at higher elevation should be staggered and there should be a minimum of four feet of compacted fill between the top of the one trench and the bottom of the next higher trench.
   d) It would be necessary to verify 90 percent relative compaction in these pits. A 24 to 72 hour delay to allow for water dissipation should be anticipated prior to additional fill placement.

Structural Rock Fills

If the materials generated for placement in structural fills contains a significant percentage of material more than six (6) inches in one dimension, then placement using conventional soil fill methods with isolated windrows would not be feasible. In such cases the following could be considered:

1. Mixes of large rock or boulders may be placed as rock fill. They should be below the depth of all utilities both on pads and in roadways and below any proposed swimming pools or other excavations. If these fills are placed within seven (7) feet of finished grade, they may affect foundation design.

2. Rock fills are required to be placed in horizontal layers that should not exceed two feet in thickness, or the maximum rock size present, which ever is less. All rocks exceeding two feet should be broken down to a smaller size, windrowed (see above), or disposed of in non-structural fill areas. Localized larger rock up to 3 feet in largest dimension may be placed in rock fill as follows:
   a) individual rocks are placed in a given lift so as to be roughly 50% exposed above the typical surface of the fill,
   b) loaded rock trucks or alternate compactors are worked around the rock on all sides to the satisfaction of the soil engineer,
   c) the portion of the rock above grade is covered with a second lift.

3. Material placed in each lift should be well graded. No unfilled spaces (voids) should be permitted in the rock fill.

Compaction Procedures

Compaction of rock fills is largely procedural. The following procedures have been found to generally produce satisfactory compaction.

1. Provisions for routing of construction traffic over the fill should be implemented.
a) Placement should be by rock trucks crossing the lift being placed and dumping at its edge.

b) The trucks should be routed so that each pass across the fill is via a different path and that all areas are uniformly traversed.

c) The dumped piles should be knocked down and spread by a large dozer (D-8 or larger suggested). Water should be applied before and during spreading.

2. Rock fill should be generously watered (sluiced)
   a) Water should be applied by water trucks to the:
      i) dump piles,
      ii) front face of the lift being placed and,
      iii) surface of the fill prior to compaction.
   b) No material should be placed without adequate water.
   c) The number of water trucks and water supply should be sufficient to provide constant water.
   d) Rock fill placement should be suspended when water trucks are unavailable:
      i) for more than 5 minutes straight, or,
      ii) for more than 10 minutes/hour.

3. In addition to the truck pattern and at the discretion of the soil engineer, large, rubber tired compactors may be required.
   a) The need for this equipment will depend largely on the ability of the operators to provide complete and uniform coverage by wheel rolling with the trucks.
   b) Other large compactors will also be considered by the soil engineer provided that required compaction is achieved.

4. Placement and compaction of the rock fill is largely procedural. Observation by trenching should be made to check:
   a) the general segregation of rock size,
   b) for any unfilled spaces between the large blocks, and
   c) the matrix compaction and moisture content.

5. Test fills may be required to evaluate relative compaction of finer grained zones or as deemed appropriate by the soil engineer.
   a) A lift should be constructed by the methods proposed, as proposed

6. Frequency of the test trenching is to be at the discretion of the soil engineer. Control areas may be used to evaluate the contractors procedures.

7. A minimum horizontal distance of 15 feet should be maintained from the face of the rock fill and any finish slope face. At least the outer 15 feet should be built of conventional fill materials.

Piping Potential and Filter Blankets
Where conventional fill is placed over rock fill, the potential for piping (migration) of the fine grained material from the conventional fill into rock fills will need to be addressed.
The potential for particle migration is related to the grain size comparisons of the materials present and in contact with each other. Provided that 15 percent of the finer soil is larger than the effective pore size of the coarse soil, then particle migration is substantially mitigated. This can be accomplished with a well-graded matrix material for the rock fill and a zone of fill similar to the matrix above it. The specific gradation of the fill materials placed during grading must be known to evaluate the need for any type of filter that may be necessary to cap the rock fills. This, unfortunately, can only be accurately determined during construction.

In the event that poorly graded matrix is used in the rock fills, properly graded filter blankets 2 to 3 feet thick separating rock fills and conventional fill may be needed. As an alternative, use of two layers of filter fabric (Mirafi 700 x or equivalent) could be employed on top of the rock fill. In order to mitigate excess puncturing, the surface of the rock fill should be well broken down and smoothed prior to placing the filter fabric. The first layer of the fabric may then be placed and covered with relatively permeable fill material (with respect to overlying material) 1 to 2 feet thick. The relative permeable material should be compacted to fill standards. The second layer of fabric should be placed and conventional fill placement continued.

Subdrainage
Rock fill areas should be tied to a subdrainage system. If conventional fill is placed that separates the rock from the main canyon subdrain, then a secondary system should be installed. A system consisting of an adequately graded base (3 to 4 percent to the lower side) with a collector system and outlets may suffice.

Additionally, at approximately every 25 foot vertical interval, a collector system with outlets should be placed at the interface of the rock fill and the conventional fill blanketing a fill slope.

Monitoring
Depending upon the depth of the rock fill and other factors, monitoring for settlement of the fill areas may be needed following completion of grading. Typically, if rock fill depths exceed 40 feet, monitoring would be recommend prior to construction of any settlement sensitive improvements. Delays of 3 to 6 months or longer can be expected prior to the start of construction.

**UTILITY TRENCH CONSTRUCTION AND BACKFILL**

Utility trench excavation and backfill is the contractors responsibility. The geotechnical consultant typically provides periodic observation and testing of these operations. While efforts are made to make sufficient observations and tests to verify that the contractors' methods and procedures are adequate to achieve proper compaction, it is typically impractical to observe all backfill procedures. As such, it is critical that the contractor use consistent backfill procedures.
Compaction methods vary for trench compaction and experience indicates many methods can be successful. However, procedures that "worked" on previous projects may or may not prove effective on a given site. The contractor(s) should outline the procedures proposed, so that we may discuss them prior to construction. We will offer comments based on our knowledge of site conditions and experience.

1. Utility trench backfill in slopes, structural areas, in streets and beneath flat work or hardscape should be brought to at least optimum moisture and compacted to at least 90 percent of the laboratory standard. Soil should be moisture conditioned prior to placing in the trench.

2. Flooding and jetting are not typically recommended or acceptable for native soils. Flooding or jetting may be used with select sand having a Sand Equivalent (SE) of 30 or higher. This is typically limited to the following uses:
   a) shallow (12 + inches) under slab interior trenches and,
   b) as bedding in pipe zone.

   The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.

3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.

4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.

5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the contractors procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this would be brought to the contractors attention.

**JOB SAFETY**

**General**

Personnel safety is a primary concern on all job sites. The following summaries are safety considerations for use by all our employees on multi-employer construction sites. On ground personnel are at highest risk of injury and possible fatality on grading construction projects. The company recognizes that construction activities will vary on each site and that job site safety is the contractor’s responsibility. However, it is imperative that all personnel be safety conscious to avoid accidents and potential injury.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of our field personnel on grading and construction projects.
1. Safety Meetings: Our field personnel are directed to attend the contractor's regularly scheduled safety meetings.

2. Safety Vests: Safety vests are provided for and are to be worn by our personnel while on the job site.

3. Safety Flags: Safety flags are provided to our field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

Test Pits Location, Orientation and Clearance

The technician is responsible for selecting test pit locations. The primary concern is the technician's safety. However, it is necessary to take sufficient tests at various locations to obtain a representative sampling of the fill. As such, efforts will be made to coordinate locations with the grading contractors authorized representatives (e.g. dump man, operator, supervisor, grade checker, etc.), and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractors authorized representative should direct excavation of the pit and safety during the test period. Again, safety is the paramount concern.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates that the fill be maintained in a drivable condition. Alternatively, the contractor may opt to park a piece of equipment in front of test pits, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits (see diagram below). No grading equipment should enter this zone during the test procedure. The zone should extend outward to the sides approximately 50 feet from the center of the test pit and 100 feet in the direction of traffic flow. This zone is established both for safety and to avoid excessive ground vibration, which typically decreases test results.
Slope Tests
When taking slope tests, the technician should park their vehicle directly above or below the test location on the slope. The contractor's representative should effectively keep all equipment at a safe operation distance (e.g. 50 feet) away from the slope during testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location.

Trench Safety
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Trenches for all utilities should be excavated in accordance with CAL-OSHA and any other applicable safety standards. Safe conditions will be required to enable compaction testing of the trench backfill.

All utility trench excavations in excess of 5 feet deep, which a person enters, are to be shored or laid back. Trench access should be provided in accordance with OSHA standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

Our personnel are directed not to enter any excavation which;
1. is 5 feet or deeper unless shored or laid back,
2. exit points or ladders are not provided,
3. displays any evidence of instability, has any loose rock or other debris which could fall into the trench, or
4. displays any other evidence of any unsafe conditions regardless of depth.
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraws and notifies their supervisor. The contractor's representative will then be contacted in an effort to effect a solution. All backfill not tested due to safety concerns or other reasons is subject to reprocessing and/or removal.

**Procedures**

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by company policy, to immediately withdraw and notify their supervisor. The contractor's representative will then be contacted in an effort to effect a solution. No further testing will be performed until the situation is rectified. Any fill placed in the interim can be considered unacceptable and subject to reprocessing, recompaction or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to technicians attention and notify our project manager or office. Effective communication and coordination between the contractors' representative and the field technician(s) is strongly encouraged in order to implement the above safety program and safety in general.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

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