LIMITED GEOTECHNICAL EVALUATION
PROPOSED PARKING LOT FOR VIRI ESTATES DEVELOPMENT
NORTHEASTERLY CORNER OF APN 148-271-11
OCEANSIDE, SAN DIEGO COUNTY, CALIFORNIA

FOR

SEKHI, LLC
629 S. TWIN OAKS VALLEY ROAD, #106
SAN MARCOS, CALIFORNIA 92078

W.O. 6912-A1-SC NOVEMBER 30, 2016

Geotechnical • Geologic • Coastal • Environmental

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November 30, 2016

W.O. 6912-A1-SC

Sekhi, LLC
629 S. Twin Oaks Valley Road, #106
San Marcos, California 92078

Attention: Mr. Brent Mitchell

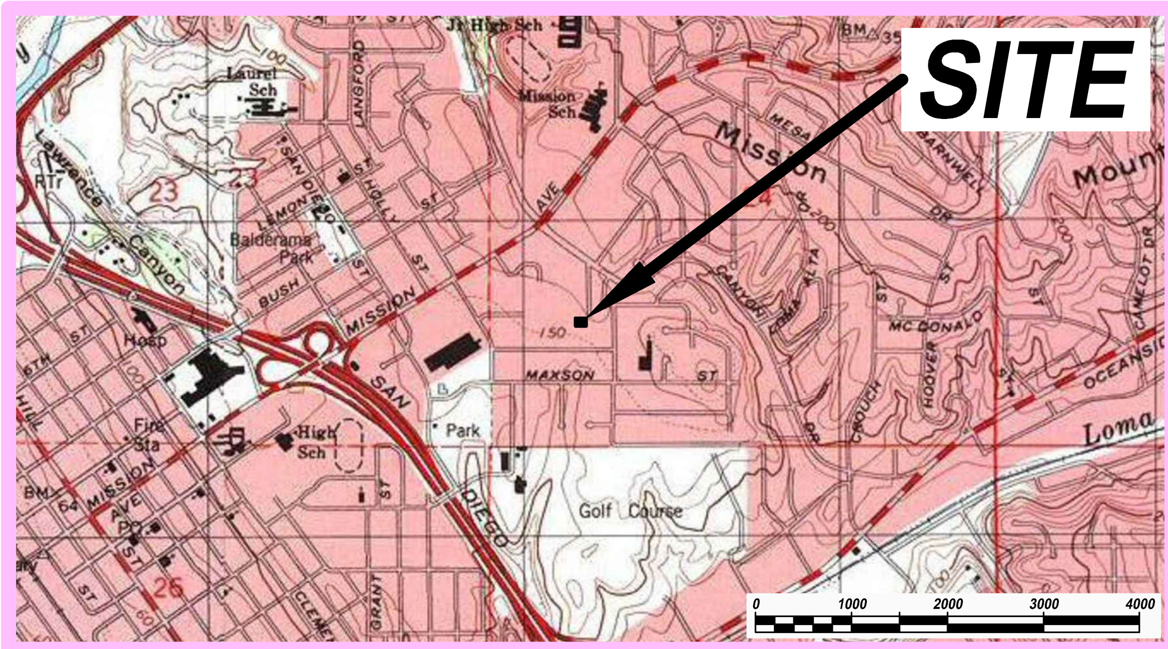
Subject: Limited Geotechnical Evaluation, Proposed Parking Lot for Viri Estates Development, Northeasterly Corner of APN 148-271-11, Oceanside, San Diego County, California

Dear Mr. Mitchell:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) is providing this summary of our limited geotechnical evaluation relative to the proposed parking lot improvements at the subject parcel. Based upon communication with you, GSI understands that the northeasterly corner of the subject parcel will be leased by the Client in order to provide additional parking area for the proposed Viri Estates project. The services GSI performed for this study included a review of the referenced documents (see Appendix), geotechnical engineering analyses, and the preparation of this summary report. Unless specifically superceded herein, the conclusions and recommendations contained in GSI (2004, 2005, 2007, and 2016) are still considered valid and applicable and should be appropriately implemented during project design and construction.

SITE DESCRIPTION/PROPOSED DEVELOPMENT

According to Exhibit “B” (Buccola Engineering, 2016), the study area, under the purview of this report, consists of approximately 8,666 square feet of relatively undeveloped land, located at the northeasterly corner of 1836 Dixie Street, Oceanside, San Diego County, California 92054 (see Figure 1, Site Location Map). The site is bounded by a church parking lot to the west and by existing residential development to the remaining quadrants. Topographically, the study area is relatively flat-lying to very gently sloping in a southwesterly direction. According to Google Earth imagery site elevations vary between approximately 155 feet and 157 feet (unknown datum) for an overall relief of about 2 feet. Site drainage appears to be accommodated by sheet-flow runoff, directed to the southwest, where it appears to discharge into the aforementioned church parking lot.



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S. San Luis Rey Quadrangle, California -- San Diego Co., 7.5 Minute, dated 1997, current, 1999.

Base Map: Google Maps, Copyright 2016 Google, Map Data Copyright 2016 Google

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SITE LOCATION MAP

Figure 1

Based on communication with the Client, GSI understands that the subject site will be prepared to receive a new parking lot equipped with asphaltic concrete (AC) pavement and lighting poles (lighting standards). Although, grading plans have not been provided for GSI review, we anticipate that minor cut and fill grading will be necessary to achieve the design grades with maximum planned cuts and fills on the order of 1 foot or less. Graded slope construction is not anticipated at this time.

SITE GEOLOGIC CONDITIONS

Based on a review of a previous subsurface study, performed on the adjacent easterly property (GSI, 2004), it is anticipated that the onsite earth materials consist of Quaternary-age colluvium overlying Quaternary-age old paralic deposits. A Quaternary-age paleosol may be discontinuously imprinted upon the old paralic deposits, and localized areas of undocumented fill may occur at the surface. In general, the colluvium is anticipated to consist of brown and red brown sandy clay, brown clayey sand, brown silty sand, and dark gray brown sandy silt that are dry to damp and soft/loose to medium stiff/medium dense.

The old paralic deposits likely consist of dry to moist and stiff to very stiff/very dense, dark red brown, red brown, yellow brown, and light red brown silty sand and brown, olive brown, and light red brown sandy clay. As observed on the adjacent easterly property, the upper approximately 1 to 1½ feet of these deposits may be weathered. The paleosol, if present, typically consists of a dark red brown and red brown sandy clay with local scattered gravels that is generally damp to moist and medium stiff to stiff. Any existing fills would likely consist of an aggregation of the above-described earth materials.

PRELIMINARY CONCLUSIONS

Based on our limited review and provided the recommendations in this report are properly incorporated into project design and construction, it is our opinion that the subject site is suitable to receive the proposed parking lot improvements, from a geotechnical perspective. The primary geotechnical concerns with respect to the proposed development and improvements are:

- Earth materials characteristics and depth to competent bearing material.
- Relatively low resistance value (R-value) of the onsite soils.
- On-going expansion and corrosion potential of site soils.
- Rock hardness/excavation feasibility
- Erosiveness of site earth materials.
- Potential for perched water during and following site development.
- Perimeter conditions and planned improvements near the property boundary.

- Monitoring of adjacent property and temporary slopes during planned excavations near the property lines.
- The control of surface and subsurface water.

The most significant findings of our investigation are summarized below.

1. Quaternary-age colluvium, the Quaternary-age paleosol, and weathered portions of the Quaternary-age old paralic deposits are considered unsuitable for the support of the proposed parking lot improvements and new planned fills. Unweathered old paralic deposits are considered suitable bearing materials. Based on the available subsurface data obtained during our previous work on the adjacent easterly property, the depth of remedial grading excavations to encounter suitable bearing materials is anticipated to range between approximately ½ foot and 4½ feet below the existing grades. However, they may occur deeper, locally. This should be considered during project planning and construction. Upon request, the depth of remedial grading excavations at the subject site can be further evaluated by site-specific subsurface exploration.

2. Expansion index (E.I.) testing, performed on samples of earth materials, collected from the adjacent easterly property, in preparation of GSI (2004), suggests that some of the onsite soils could have expansion indices ranging between 76 and 126. This corresponds to medium and high expansion potentials, respectively. Previous Atterberg limits testing on these samples indicates Plasticity Indices (P.I.s) ranging between 28 and 40. Based on these test results, the onsite soils are considered detrimentally expansive, on a preliminary basis.

3. Previous laboratory testing, performed to evaluate the corrosiveness of a representative sample of soils, collected from the adjacent easterly property, in preparation of GSI (2004), suggests that the onsite soil is mildly alkaline with respect to soil acidity/alkalinity; corrosive to exposed, buried metals when saturated; presents a negligible ("not applicable" per American Concrete Institute [ACI] 318-11) sulfate exposure to concrete; and non-detectable concentrations of soluble chlorides. It should be noted that GSI does not consult in the field of corrosion engineering. Thus, the client and project architect should agree on the level of corrosion protection required for the project and seek consultation from a qualified corrosion consultant, as warranted.

4. Previous resistance value (R-value) testing, performed on representative bulk soil samples, collected from nearby Dixie and Grace Streets, in preparation of GSI (2007), indicated R-value test results ranging between 9 and 13. Pavement design for the proposed parking lot should consider the potential for low subgrade R-values, which will likely necessitate relatively thick pavement sections.

5. During our subsurface exploration, performed in preparation of GSI (2004), a dense, cemented hardpan was encountered within some of the test pits, excavated on the adjacent easterly property. This hardpan may create difficult excavations with lightweight excavation equipment such as backhoes and mini-excavators, and possibly heavier excavators. However, based on our experience, it should be rippable with a Caterpillar D-9L bulldozer. Given the potential for difficult excavation, the Client may consider overexcavating proposed parking lot areas, receiving underground utilities, to at least 1 foot below the lowest underground utility invert and replacing the overexcavated materials with engineered fill during grading to help facilitate underground utility construction. This is not a geotechnical requirement, however.

6. The onsite soils are considered erosive. Therefore, engineered surface drainage controls are recommended. Vegetative and surface coverings (i.e., pavements and hardscape) will also provide protection from surficial erosion. Temporary erosion control measures should be provided until vegetation covering is well established and planned pavements are constructed.

7. No evidence of a high regional groundwater table nor perched water was observed during the subsurface exploration performed in preparation of GSI (2004), on the adjacent, easterly property. Similar conditions are anticipated within the subject site. Due to the nature of site earth materials, there is a potential for perched water to occur both during and following site development. This potential should be disclosed to all interested/affected parties. Should perched water conditions be encountered, this office could provide recommendations for mitigation. Typical mitigation includes subdrainage system, cut-off barriers, etc.

8. Removal and recompaction of potentially compressible soils below a 1:1 (horizontal: vertical [h:v]) projection down from the bottom outside of planned settlement-sensitive improvements and new planned fills along the perimeter of the site will be limited due to boundary restrictions. As such, any settlement-sensitive improvement located above a 1:1 (h:v) projection from the bottom outboard edge of the remedial grading excavation at the property line would require deepened foundations below this plane, additional reinforcement, or would retain some potential for distress and therefore, a reduced serviceable life. On a preliminary basis, any planned settlement-sensitive improvements located within approximately ½-foot and 4½ feet from the property lines would require deepened foundations or additional reinforcement by means of ground improvement or specific structural design. This should be considered during project design. Light standard foundations near the property margins will likely need to be deepened into unweathered old paralic deposits.

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

All earthwork should conform to the guidelines presented in the 2013 CBC (CBSC, 2013), the requirements of the City of Oceanside, and the General Earthwork and Grading Guidelines presented in Appendix C, except where specifically superceded in the text of this report. Prior to earthwork, a GSI representative should be present at the preconstruction meeting to provide additional earthwork guidelines, if needed, and review the earthwork schedule. This office should be notified in advance of any fill placement, supplemental regrading of the site, or backfilling underground utility trenches after rough earthwork has been completed. This includes grading for the proposed surface improvements.

During earthwork construction, all site preparation and the general grading procedures of the contractor should be observed and the fill selectively tested by a representative(s) of GSI. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and, if warranted, modified and/or additional recommendations will be offered. All applicable requirements of local and national construction and general industry safety orders, the Occupational Safety and Health Act (OSHA), and the Construction Safety Act should be met. It is the onsite general contractor's and individual subcontractors' responsibility to provide a safe working environment for our field staff who are onsite. GSI does not consult in the area of safety engineering. It is also the responsibility of the contractor to provide protection of their work product. Surface drainage should be directed away from open excavations.

Site Preparation

All existing improvements, vegetation, and deleterious debris should be removed from the site, prior to the start of construction, if they are located in areas of proposed earthwork.

Any remaining cavities resulting from the removal of any existing improvements should be observed by the geotechnical consultant. Mitigation of cavities would likely include removing any potentially compressible soils to expose unweathered old paralic deposits and then backfilling the excavation with a controlled engineered fill or soils that have been moisture conditioned to optimum moisture content and compacted to at least 90 percent of the laboratory standard (ASTM D 1557).

It is possible that an onsite sewage disposal system may be present. Should such structures be encountered during earthwork, this office should be contacted to provide recommendations for removal and disposal.

Remedial Excavation - Removal and Recompaction of Potentially Compressible Earth Materials

Potentially compressible undocumented fill, colluvium, and weathered old paralic deposits/paleosol should be removed to expose unweathered old paralic deposits. Following remedial excavation, these soils should be cleaned of any vegetation and deleterious debris, moisture conditioned to at least the soil's optimum moisture content, and then be reused as engineered fill in accordance with the recommendations in the "Fill Placement"section below. Based on the available subsurface data, excavations necessary to remove unsuitable soils are anticipated to range between approximately ½ foot and 4½ feet below existing grades. The potential to encounter thicker sections of unsuitable soils that require deeper remedial grading excavations, than stated above, cannot be precluded and should be anticipated. Potentially compressible soils should be removed below a 1:1 (h:v) projection down from the bottom, outboard edge of any settlement-sensitive improvement or limits of planned fill unless constrained by property lines or onsite improvements that need to remain in serviceable use both during and following site development. Remedial grading excavations should be observed by the geotechnical consultant prior to scarification and fill placement. Once observed and approved, the bottom of the remedial grading excavation should be scarified at least 6 to 8 inches, moisture conditioned to at least the soil's optimum moisture content, and then be recompacted in accordance with the recommendations in the "Fill Placement"section below.

Perimeter Conditions

It should be noted that the 2013 CBC (CBSC, 2013) indicates that removals of unsuitable soils be performed across all areas to be graded, under the purview of the grading permit (if required), not just within the influence of the planned structures. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas. This zone would be approximately equal to the depth of removals, if removals cannot be performed onsite or offsite. In general, any planned improvement located above a 1:1 (h:v) projection up from the bottom, outboard edge of the remedial grading excavation at the property boundaries or adjacent to existing improvements that need to remain and serviceable use both during and following site development would be affected by perimeter conditions. On a preliminary basis, any planned settlement-sensitive improvements located within approximately ½ foot and 4½ feet from the property lines would require deepened foundations or additional reinforcement by means of ground improvement or specific structural design, for the perimeter conditions discussed above. Otherwise, these improvements may be subject to distress and a reduced serviceable life span. This would require proper disclosure to all interested/affected parties should this condition exist at the conclusion of grading. If remedial grading cannot be performed below a 1:1 (h:v) projection down from the bottom outboard edges of the planned light standard foundations along property lines, the foundation would need to extend into unweathered old paralic deposits for vertical and lateral bearing support.

Fill Placement

Following scarification of the bottom of the remedial grading excavations, the reused onsite soils and import (if necessary) should be placed in ±6- to ±8-inch lifts, cleaned of vegetation and debris, moisture conditioned to at least the soil's optimum moisture content, and compacted to achieve a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557). Fill materials should not contain rock constituents greater than 12 inches in any dimension. Keying and benching is recommended during the placement of fill on surfaces with gradients steeper than 5:1 (h:v).

Import Soils

If import fill is necessary, a sample of the soil import should be evaluated by this office prior to importing, in order to assure compatibility with the onsite soils and the recommendations presented in this report. If non-manufactured materials are used, environmental documentation for the export site should be provided for GSI review. At least five business days of lead time should be allowed by builders or contractors for proposed import submittals. This lead time will allow for environmental document review, particle size analysis, laboratory standard, expansion testing, and blended import/native characteristics as deemed necessary. If earthwork mitigation of expansive soils is not incorporated into the grading of the site import soils may have an expansion index of 90 or less. Import for the use of expansive soil mitigation should be non-detrimentally expansive (i.e., E.I. less than 21 and P.I. less than 15). The use of subdrains at the bottom of the fill cap may be necessary, and may be subsequently recommended based on compatibility with onsite soils.

Temporary Slopes

Temporary slopes for excavations greater than 4 feet, but less than 20 feet in overall height should conform to CAL-OSHA and/or OSHA requirements for Type "B" soils, provided water or seepage and/or running sands are not present. Temporary slopes, up to a maximum height of ±20 feet, may be excavated at a 1:1 (h:v) gradient, or flatter, provided groundwater and/or running sands are not exposed. Construction materials, soil stockpiles, and heavy equipment should not be stored and/or operated within 'H' of any temporary slope where 'H' equals the height of the temporary slope. All temporary slopes should be observed by a licensed engineering geologist and/or geotechnical engineer prior to worker entry into the excavation. Based on the exposed field conditions, inclining temporary slopes to flatter gradients or the use of shoring may be necessary if adverse conditions are observed. If adverse conditions are exposed or if temporary slopes conflict with property boundaries or existing improvements to remain following development, shoring or alternating slot excavations may be necessary. The need for shoring or alternating slot excavations could be further evaluated during the grading plan review stage.

Excavation Observation and Monitoring (All Excavations)

When excavations are made adjacent to an existing improvement (i.e., utility, wall, road, building, etc.) there is a risk of some damage even if a well designed system of excavation is planned and executed. We recommend, therefore, that a systematic program of observations be made before, during, and after construction to determine the effects (if any) of construction on existing improvements.

We believe that this is necessary for two reasons: First, if excessive movements (i.e., more than ½ inch) are detected early enough, remedial measures can be taken which could possibly prevent serious damage to existing improvements. Second, the responsibility for damage to the existing improvement can be determined more equitably if the cause and extent of the damage can be determined more precisely.

Monitoring should include the measurement of any horizontal and vertical movements of the existing structures/improvements. Locations and type of the monitoring devices should be selected prior to the start of construction. The program of monitoring should be agreed upon between the project team, the site surveyor and the Geotechnical Engineer-of-Record, prior to excavation.

Reference points on existing walls, buildings, and other settlement-sensitive improvements. These points should be placed as low as possible on the wall and building adjacent to the excavation. Exact locations may be dictated by critical points, such as bearing walls or columns for buildings; and surface points on roadways or curbs near the top of the excavation.

For a survey monitoring system, an accuracy of a least 0.01 foot should be required. Reference points should be installed and read initially prior to excavation. The readings should continue until all construction below ground has been completed and the permanent backfill has been brought to final grade.

The frequency of readings will depend upon the results of previous readings and the rate of construction. Weekly readings could be assumed throughout the duration of construction with daily readings during rapid excavation near the bottom of the excavation. The reading should be plotted by the Surveyor and then reviewed by the Geotechnical Engineer. In addition to the monitoring system, it would be prudent for the Geotechnical Engineer and the Contractor to make a complete inspection of the existing structures both before and after construction. The inspection should be directed toward detecting any signs of damage, particularly those caused by settlement. Notes should be made and pictures should be taken where necessary.

Observation

It is recommended that all excavations be observed by the Geologist and/or Geotechnical Engineer. Any fill which is placed should be approved, tested, and verified if used for engineered purposes. Should the observation reveal any unforseen hazard, the Geologist or Geotechnical Engineer will recommend treatment. Please inform GSI at least 24 hours prior to any required site observation.

PRELIMINARY ASPHALTIC CONCRETE OVER AGGREGATE BASE SECTION

The recommended asphaltic concrete over aggregate base sections (AC/AB) within the proposed parking lot drive lanes and parking stalls are provided herein. The recommended AC/AB pavement sections were developed through pavement design analyses incorporating the Traffic Index (TI) of the parking lot drive lanes and parking stalls and consider the subgrade R-value test results obtained during the GSI (2007) study. For the parking lot drive lanes and parking stalls, GSI has assumed TI values of 5.0 and 4.5, respectively. A subgrade R-value of 5.0 was considered in the analyses, per City of Oceanside (1992) guidelines. Preliminary AC/AB pavement sections are provided in the table below. Final pavement sections should be based upon the results of R-value tests, performed at the conclusion of grading and underground utility installation.

TRAFFIC AREA	TRAFFIC INDEX	SUBGRADE R-VALUE	AC THICKNESS (INCHES)	AGGREGATE BASE THICKNESS(2) (INCHES)
Parking Lot Drive Lanes	5.0	5	3.0(1)	10.0
Parking Stalls	4.5	5	3.0(1)	8.5
Parking Lot Drive Lanes Alternative Section	5.0	5	4.0	7.5
Parking Stalls Alternative Section	4.5	5	4.0	6.0
(1) City of Oceanside minimum thickness (2) Denotes Class 2 Aggregate Base (R$\geq$78, SE $\geq$22)				

All pavement installation, including preparation and compaction of subgrade, compaction of base material, and placement and rolling of asphaltic concrete, should be done in accordance with City of Oceanside guidelines and under the observation and testing services provided by the project geotechnical engineer and/or the City.

The recommended pavement sections provided above are intended as minimum guidelines. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the T.I. used for design, increased maintenance and repair could be required for the pavement section. Consideration should be given to the increased potential for distress from overuse of paved street areas by heavy equipment and/or construction related heavy traffic (e.g., concrete trucks, loaded supply trucks, etc.), particularly when the final section is not in place (i.e., topcoat). Best management construction practices should be followed at all times, especially during inclement weather.

PAVEMENT GRADING RECOMMENDATIONS

General

All section changes should be properly transitioned. If adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. A GSI representative should be present for the preparation of subgrade, base rock, and asphalt concrete.

Subgrade

Within street and parking areas, all surficial deposits of loose soil material should be removed and recompacted as recommended. After the loose soils are removed, the bottom is to be scarified to a depth of at least 6 inches, moisture conditioned to at least optimum moisture content, and compacted to 95 percent of the maximum laboratory density (ASTM D-1557) or the City minimum, as determined by ASTM D-1557. If highly expansive soils are present, the subgrade should be compacted 90 percent of the laboratory standard.

Deleterious material, excessively wet or dry pockets, concentrated zones of oversized rock fragments, and any other unsuitable materials encountered during grading should be removed. The compacted fill material should then be brought to the elevation of the proposed subgrade for the pavement. The subgrade should be proof-rolled in order to ensure a uniform firm and unyielding surface. All grading and fill placement should be observed by the project soil engineer and/or his representative.

Aggregate Base Rock

Compaction tests are required for the recommended base section. Minimum relative compaction required will be 95 percent of the laboratory maximum density as determined by ASTM test method D-1557 and/or Caltrans Test Method Number California 216. Base aggregate should be in accordance with the Caltrans Class 2 base rock (minimum R-value=78).

Paving

Prime coat may be omitted if all of the following conditions are met:

1. The asphalt pavement layer is placed within two weeks of completion of base and/or subbase course.

2. Traffic is not routed over completed base before paving

3. Construction is completed during the dry season of May through October.

4. The base is kept free of debris prior to placement of asphaltic concrete.

If construction is performed during the wet season of November through April, prime coat may be omitted if no rain occurs between completion of base course and paving <u>and</u> the time between completion of base and paving is reduced to three days, provided the base is free of loose soil or debris. Where prime coat has been omitted and rain occurs, traffic is routed over base course, or paving is delayed, measures shall be taken to restore base course, and subgrade to conditions that will meet specifications as directed by the soil engineer.

Drainage

Positive drainage should be provided for all surface water to drain towards the curb and gutter, or to an approved drainage channel. Positive site drainage should be maintained at all times. Water should not be allowed to pond or seep into the ground. If planters or landscaping are adjacent to paved areas, measures should be taken to minimize the potential for water to enter the pavement section, such as thickened edges, cut-offs wall, french drains, etc. GSI recommends that drainage, within the project area, should be designed to <u>not</u> saturate subgrade soils within the proposed parking lot. The Client should consider the use of thickened curbs, cut-off walls, or french drains along the perimeter of the parking lot in order to reduce the potential for saturation of subgrade soils. The thickened curbs, cut-off walls, or french drains should extend at least 1 foot below the subgrade elevation. Available R-value test data indicates that the subgrade soils, underlying these streets, may deflect considerably under load when wet, potentially resulting in pavement distress and reduced serviceable life. If french drains are used, the pipe should be tightlined to a suitable outlet as approved by the City of Oceanside. GSI will provide additional recommendations for french drains if this option is requested.

PORTLAND CEMENT CONCRETE CURBS, GUTTERS, AND SIDEWALKS

Per City of Oceanside requirements, all Portland Cement Concrete (PCC) curbs, gutters (including cross gutters), and sidewalks shall receive a minimum of 6 inches of Class 2 aggregate base placed above a properly prepared and compacted subgrade. Due to the expansiveness of the onsite soils, the subgrade supporting the aforementioned PCC improvements, should be moisture conditioned to at least the soil's optimum moisture content, and then be compacted to at least 90 percent of the laboratory standard (ASTM D 1557).

LIGHTING POLE STANDARDS

San Diego Regional Standard Drawing SDE-101 (City of San Diego Public Works Department, 2016) provides minimum guidelines of street lighting standards. The actual design of lighting pole standards should be provided by the project civil engineer based on the type of lighting pole to be used and anticipated loading factors, and consider the aforementioned guidelines. Prior to construction, the design of the lighting pole standards should be reviewed by GSI.

LIMITATIONS

The conclusions and recommendations are professional opinions. These opinions have been derived in accordance with current standards of practice, and no warranty, either express or implied, is given. Standards of practice are subject to change with time. GSI assumes no responsibility or liability for work or testing performed by others, or their inaction; or work performed when GSI is not requested to be onsite, to evaluate if our recommendations have been properly implemented. Use of this report constitutes an agreement and consent by the user to all the limitations outlined above, notwithstanding any other agreements that may be in place. In addition, this report may be subject to review by the controlling authorities. Thus, this report brings to completion our scope of services for this portion of the project.

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

Respectfully submitted,

GeoSoils, Inc.

John P. Franklin
Engineering Geologist, CEG 1340

David W. Skelly
Civil Engineer, RCE 47857

Ryan B. Boehmer
Project Geologist

RBB/JPF/DWS/jh

Attachment: Appendix - References

Distribution: (1) Addressee (via US Mail and email)
(3) Buccola Engineering, Attn: Mr. Phil Buccola (3 wet signed and email)

APPENDIX

REFERENCES

Buccola Engineering, Inc., Exhibits “A” and “B”, 2 sheets, 50-scale, plot dated September 16.

City of San Diego Public Works Department, 2016, Standard drawings for public works construction.

City of Oceanside Public Works Department - Engineering Division, 1992, Engineers design and processing manual, revision dated August 26.

GeoSoils, Inc., 2016, Update geotechnical evaluation, proposed convalescent development, Viri Estates, 1914 Dixie Street, Oceanside, San Diego County, California, APN 148-271-10, W.O. 6912-A-SC, revision dated June 30.

_____, 2007, Evaluation of existing pavement section, portions of Dixie and Grace Streets, Dixie Street Project, Oceanside, San Diego County, California, W.O. 4396-E-SC, dated May 2.

_____, 2005, Response to review comments, Review memoranda dated January 10 and January 27, 2005, 1914 through 1918 Dixie Street, Oceanside, San Diego County, California, W.O. 4396-A1-SC, dated April 27.

_____, 2004, Preliminary geotechnical evaluation, 1914 through 1918 Dixie Street, Oceanside, San Diego County, California, W.O. 4396-A-SC, dated August 9.

UPDATE GEOTECHNICAL EVALUATION
PROPOSED CONVALESCENT DEVELOPMENT
VIRI ESTATES, 1914 DIXIE STREET
OCEANSIDE, SAN DIEGO COUNTY, CALIFORNIA
APN 148-271-10

FOR

SEKHI, LLC
629 S. TWIN OAKS VALLEY ROAD, #106
SAN MARCOS, CALIFORNIA 92078

W.O. 6912-A-SC JUNE 24, 2015
REVISED JUNE 30, 2015

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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Revised June 30, 2015
June 24, 2015

W.O. 6912-A-SC

Sekhi, LLC
629 S. Twin Oaks Valley Road, #106
San Marcos, California 92078

Attention: Mr. Brent Mitchell

Subject: Update Geotechnical Evaluation, Proposed Convalescent Development, Viri Estates, 1914 Dixie Street, Oceanside, San Diego County, California, APN 148-271-10

Dear Mr. Mitchell:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) is pleased to present the results of our update geotechnical evaluation of the subject site. The primary purpose of our study was to evaluate the site geologic and geotechnical conditions relative to the proposed convalescent development shown on the "Tentative Parcel Map and Development Plan" prepared by Buccola Engineering, Inc. ([BEI], 2015 [see Appendix A]), and to provide preliminary recommendations for earthwork and design criteria for foundations, slab-on-grade floors, walls, pavements, and other improvements possibly applicable to the project. A secondary purpose of our study was to review and update the existing geotechnical reports GSI prepared for a previous residential development concept at the site (GSI; 2004, 2005, and 2007) to specifically address the currently proposed development shown on BEI (2015) and the requirements of the 2013 edition of the California Building Code ([2013 CBC], California Building Standards Commission [CBSC], 2013), and current standards of practice. Unless specifically superceded herein, the conclusions and recommendations contained in GSI (2004, 2005, and 2007) are still considered valid and applicable and should be appropriately implemented during project design and construction. GSI (2004, 2005, and 2007) are provided on the compact disc in Appendix A of this report.

EXECUTIVE SUMMARY

Based upon our field exploration, geologic, and geotechnical engineering analysis, the proposed convalescent development appears feasible from a soils engineering and geologic viewpoint, provided that the recommendations presented in the text of this report are properly incorporated into the design and construction of the project. The most significant elements of our study are summarized below:

- As previously indicated in GSI (2004), site earth materials generally consist of Quaternary-age colluvium (i.e., topsoil) overlying Quaternary-age terrace deposits (currently referred to as Quaternary-age old paralic deposits). An approximately 1- to 1½-foot thick paleosol (i.e., ancient soil), imprinted upon the old paralic deposits, occurs locally. In addition, the upper approximately 1 foot to 1½ feet of the old paralic deposits is locally weathered. The colluvium, paleosol, and weathered old paralic deposits are considered potentially compressible in their existing state, and therefore require mitigation, where located within the influence of the proposed settlement-sensitive improvements (i.e., buildings, underground utilities, walls, pavements, hardscape, etc) and new planned fills. The unweathered old paralic deposits are considered formational materials at the site and are considered suitable bearing materials.

- Remedial excavations for the treatment of the potentially compressible colluvium and paleosol are anticipated to extend approximately ±½ foot to ±4½ feet below existing grades.

- It should be noted that the 2013 CBC (CBSC, 2013) indicates that the removal and recompaction of unsuitable soils be performed across all areas to be graded, under the purview of a grading permit (if required), and not just within the influence of the proposed improvements. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas. This zone would be approximately equal to the depth of removals, if removals cannot be performed onsite or offsite. In general, any planned settlement-sensitive improvement located above a 1:1 (horizontal:vertical [h:v]) projection up from the bottom, outboard edge of the remedial grading excavation at the property boundary would be affected by perimeter conditions. On a preliminary basis, the available subsurface data suggests that any planned settlement-sensitive improvement located within approximately ½ foot to 4½ feet from the property lines would require deepened foundations or additional reinforcement by means of ground improvement or specific structural design. Otherwise, these improvements may be subject to distress and a reduced serviceable life span. This should be disclosed to all interested/affected parties should this condition exist at the conclusion of grading.

- Previous expansion index (E.I.) testing, performed on collected samples of the onsite earth materials during the preparation of GSI (2004), indicates expansion indices ranging between 76 and 126. This corresponds to medium and high expansion potentials, respectively. Previous Atterberg limits testing on these samples indicates plasticity indices (P.I.s) ranging between 28 and 40. Based on these test results, the onsite soils meet the criteria of detrimentally expansive soils, as indicated in Section 1803.5.3. of the 2013 CBC (CBSC, 2013). In order to comply with 2013 CBC requirements for the mitigation of expansive soils, the proposed structures will need specific foundation and slab-on-grade design that will tolerate the shrink/swell effects of expansive soils (see Section 1808.6.2 of the 2013 CBC). Alternatively, expansive soils within the influence of the proposed structures may be removed and replaced with very low expansive soils (E.I. less than 21) with a plasticity index (P.I.) less than 15 (see Section 1808.6.3 of the 2013 CBC). These mitigative scenarios are further discussed herein.

- Previous corrosion testing performed on a representative sample of the onsite soils during the preparation of GSI (2004) indicates site soils are mildly alkaline with respect to soil acidity/alkalinity; corrosive to exposed buried metals when saturated; present negligible (not applicable) sulfate exposure to concrete; and non-detectable concentrations of soluble chlorides. GSI does not consult in the field of corrosion engineering. Therefore, the Client, project architect, project structural engineer, and project civil engineer should decide on the level of corrosion protection required for the project and then seek consultation from a qualified corrosion engineer, as warranted.

- Subsurface water was not encountered within subsurface explorations performed during the preparation of GSI (2004 and 2005) to the depths explored. As such, groundwater is not anticipated to significantly affect the planned improvements. Perched water may be encountered during development or in the future, along zones of contrasting permeability and/or density. This potential should be disclosed to all interested/affected parties.

- Our evaluation indicates there are no known <u>active</u> faults crossing the site. According to regional geologic mapping by Kennedy and Tan (2005), a buried or concealed, northeast-trending fault transects the subject site. This fault is shown to displace sediments within the Santiago Formation in exposures to the northeast of the site. However, it is not shown to displace the old paralic deposits underlying the site. The estimated age of the old paralic deposits ranges between 220,000 to 413,000 years before present (Kennedy and Tan, 2005). Thus, the fault is clearly pre-Holocene in age.

- Given its relatively gentle sloping topography and soil conditions, the site is not considered to be susceptible to deep-seated landslides.

- Owing to the depth to groundwater and the dense nature of the old paralic deposits, the potential for the site to be adversely affected by liquefaction/lateral spreading is considered low.

- A dense, cemented hardpan was encountered within some of our subsurface explorations. This hardpan may create difficult excavations with lightweight excavation equipment such as backhoes and mini-excavators, and possibly heavier excavators. However, based on our experience, it should be rippable with a Caterpillar D-9L bulldozer. Given the potential for difficult excavation, the client may considered overexcavating street/driveway areas to at least 1 foot below the lowest underground utility invert and replacing the overexcavated materials with engineered fill during grading to help facilitate underground utility construction.

- Excavations for planned retaining wall construction near the northerly, easterly, and westerly property lines may require shoring for the protection of offsite property. It is recommended to perform surveys of offsite improvements prior to completing planned excavations and monitoring of these excavations during construction. There may be insufficient room to construct geogrid-reinforced retaining walls.

- Planned retaining walls and privacy walls located below a 1:1 (h:v) plane down from the bottom outboard edge of offsite improvements including walls and structures should be designed for surcharge. In addition, traffic surcharge should be considered in the design of planned retaining walls if vehicular/traffic loads will occur within the aforementioned plane.

- The seismic acceleration values and design parameters provided herein should be considered during the design of the proposed development. The adverse effects of seismic shaking on the structure(s) will likely be wall cracks, some foundation/slab distress, and some seismic settlement. However, it is anticipated that the proposed structures will be repairable in the event of the design seismic event. This potential should be disclosed to all interested/affected parties.

- Additional adverse geologic features that would preclude project feasibility were not encountered, based on the available data.

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

Respectfully submitted,

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UPDATE GEOTECHNICAL EVALUATION
PROPOSED CONVALESCENT DEVELOPMENT
VIRI ESTATES, 1914 DIXIE STREET
OCEANSIDE, SAN DIEGO COUNTY, CALIFORNIA
APN 148-271-10

SCOPE OF SERVICES

The scope of our services has included the following:

1. A review of readily available published geologic literature, the existing geotechnical reports for the site (GSI; 2004, 2005, and 2007), aerial photographs, and maps of the site vicinity (see Appendix A).

2. Performed updated general areal geologic and seismic hazards evaluations (see Appendix C).

3. Analyzed existing field and laboratory data relative to the proposed development.

4. Preparation of this summary report and accompaniments.

PROPOSED DEVELOPMENT

Based on our review of BEI (2015), GSI understands that proposed convalescent development includes removing the existing concrete slabs and foundations, and preparing the site to receive four (4) multi-unit structures with associated underground utility, wall, pavement, and hardscape improvements. Minor cut and fill grading will be necessary to achieve the design grades. BEI (2015) indicates maximum cuts and fills on the order of 6 feet and 1 foot, respectively. Graded 2:1 (h:v) or flatter cut and fill slopes, and retaining walls will be utilized to transition grade differentials. The maximum height of the planned cut and fill slopes are shown as approximately 1½ feet and 1 foot, respectively. The maximum retaining wall height will be approximately 6 feet. GSI anticipates that the proposed structures will be two to three stories in height and will utilize concrete slab-on-grade floors. Building loads are currently unknown. The onsite driveway will be surfaced with brick pavers. Portland Cement Concrete (PCC) is to be utilized for the onsite sidewalks. The depths of planned underground utilities is currently unknown.

PROJECT GEOTECHNICAL BACKGROUND

Beginning in late July 2004, GSI evaluated the site for a previously proposed residential development concept. Our initial study include the excavation of six (6) exploratory test pits with a rubber-tire backhoe, laboratory testing of soil samples collected during our field exploration, geotechnical engineering analysis, and the preparation of a summary report presenting our findings, conclusions, and recommendations (GSI, 2004). The most significant findings reported in GSI (2004) included the following:

- Potentially compressible earth materials extending to approximately ½ foot to 4½ feet below the existing grades.

- Medium to highly expansive onsite soils conditions.

- The occurrence of a cemented hardpan within the old paralic deposits that may create difficulty during excavation with relatively lightweight excavation equipment.

- The potential for the site to experience moderate to strong ground shaking from regional seismic activity.

- The occurrence of soils with low saturated resistivity (i.e., corrosive soils).

In 2005, GSI responded to City of Oceanside geotechnical review comments and provided update geotechnical conclusions and recommendations. As part of our response/update (GSI, 2005), we reviewed the most current tentative map. Based on our review, additional geotechnical concerns included, perimeter conditions, planned excavations near adjoining properties, and the design of pervious pavements for onsite street areas.

In 2007, GSI performed an evaluation of existing asphaltic concrete (AC) pavement sections within the Dixie Street and Grace Street right-of-ways. This study included coring through the existing AC surface and exploring the subsurface with seven (7) shallow hand-auger borings. Samples of the existing subgrade materials were tested in the laboratory to evaluate subgrade resistance values (R-values). Based on our findings, GSI concluded in GSI (2007) that the existing AC pavement sections within Dixie and Grace Streets did not meet minimum City of Oceanside criteria considering the traffic index and the low subgrade R-values obtained from the laboratory testing. As such, we recommended that the existing AC pavement be removed and the street areas be graded to receive a new AC over Class II aggregate base sections. GSI further recommended that existing undocumented fill in the street areas be remediated during grading.

REGIONAL GEOLOGY

The subject property lies within the coastal plains physiographic region of the Peninsular Ranges Geomorphic Province of southern California. This region consists of dissected, mesa-like terraces that transition inland to rolling hills. The encompassing Peninsular Ranges Geomorphic Province is characterized as elongated mountain ranges and valleys that trend northwesterly (Norris and Webb, 1990). This geomorphic province extends from the base of the east-west aligned Santa Monica - San Gabriel Mountains, and continues south into Baja California. The mountain ranges within this province are underlain by basement rocks consisting of pre-Cretaceous metasedimentary rocks, Jurassic metavolcanic rocks, and Cretaceous plutonic (granitic) rocks.

In the San Diego region, deposition occurred during the Cretaceous Period and Cenozoic Era in the continental margin of a forearc basin. Sediments, derived from Cretaceous-age plutonic rocks and Jurassic-age volcanic rocks, were deposited during the Tertiary Period (Eocene-age) into the narrow, steep, coastal plain and continental margin of the basin. These rocks have been uplifted, eroded, and deeply incised. During early Pleistocene time, a broad coastal plain was developed from the deposition of marine terrace deposits (currently termed "paralic deposits"). During mid to late Pleistocene time, this plain was uplifted, eroded and incised. Alluvial deposits have since filled the lower valleys, and young marine sediments are currently being deposited/eroded within coastal and beach areas. Regional geologic mapping by Kennedy and Tan (2005) indicate the site is underlain by late to middle Pleistocene-age old paralic deposits (formerly termed "terrace deposits" on older geologic maps).

SITE GEOLOGIC UNITS

The site geologic units encountered, during our 2004 subsurface investigation and site reconnaissance, included Quaternary-age colluvium (i.e., topsoil), a discontinuous Quaternary-age paleosol, and Quaternary-age old paralic deposits (Kennedy and Tan, 2005). The earth materials are generally described below from the youngest to the oldest. The distribution of these materials is shown on Plate 1 (Geotechnical Map).

Quaternary Colluvium (Not Mapped)

Based on the available data, colluvium mantles the entire site and generally consists of approximately ½-foot to 2 feet of brown and red brown sandy clay, brown clayey sand, brown silty sand, and dark gray brown sandy silt. The colluvium was dry to damp and soft/loose to medium stiff/medium dense. Based on the porous nature and low relative density of these surficial soils, it is GSI's opinion that the colluvium is unsuitable for the support of settlement-sensitive improvements and/or engineered fill in its existing state, and will require removal and recompaction, if settlement-sensitive improvements and/or engineered fill are proposed within its influence.

Quaternary Paleosol (Not Mapped)

GSI observed a discontinuous paleosol developed upon the old paralic deposits in the test pits completed in 2004. This relict soil horizon generally consisted of a dark red brown and red brown sandy clay with local scattered gravels that was generally damp to moist and medium stiff to stiff. As observed in the test pits the thickness of the paleosol was on the order of 1 foot to 1½ feet. The paleosol contained local porosity and therefore is considered non-uniform. As such, it should not be used to support settlement-sensitive improvements and engineered fill without mitigation.

Quaternary Old Paralic Deposits (Map Symbol - Qop)

Quaternary old paralic deposits were encountered at depths ranging between approximately ½-foot to 3 feet below the existing grades. As observed in the test pits completed in 2004, the upper approximately 1 to 1½ feet of these deposits were weathered. The old paralic deposits consisted of dark red brown, red brown, yellow brown, and light red brown silty sand and brown, olive brown, and light red brown sandy clay. The old paralic deposits were dry to moist and stiff to very stiff/very dense. The unweathered old paralic deposits which occur at depths ranging between approximately ½ foot to 4½ feet below the existing grades, are considered suitable for the support of settlement-sensitive improvements and/or engineered fill in their existing state. The old paralic deposits exhibited a discontinuous, well-cemented hardpan. Based on our experience with other projects in the immediate vicinity, this hardpan is believed to be on the order of ±1 to ±2 feet thick, and most likely will present difficulty during underground utility excavations if relatively lightweight excavation equipment (i.e., rubber-tire backhoe and mini-excavator) is used. However, the hardpan is generally considered to be rippable with heavy grading equipment (Caterpillar D-9L bulldozer or equivalent). Based on our review of Weber (1984) and Tan and Kennedy (1996), these deposits appear to be early to mid-Quaternary in age and are approximately ±40 feet in thickness in the vicinity of the site.

GROUNDWATER

Subsurface water was not encountered within the property during field work performed in preparation of GSI (2004). Subsurface water is not anticipated to adversely affect site development, provided that the recommendations contained in this report are incorporated into final design and construction. These observations reflect site conditions at the time of our investigation and do not preclude future changes in local groundwater conditions from excessive irrigation, precipitation, or other factors that were not obvious at the time of our investigation. Based on GSI's experience with other sites in the vicinity, a perched groundwater table typically occurs near the contact between the old paralic deposits and the underlying Tertiary Santiago Formation. According to Kennedy and Tan (1996 and 2005), this geologic contact is located to the east of the site at approximate elevations of 120 to 130 feet Mean Sea Level (MSL). Therefore, a perched groundwater table may be located approximately 20 to 30 feet below the lowest site elevation. The regional groundwater table is anticipated to be near MSL (approximately 150 feet below the lowest site elevation). Should this perched water table be present, it should not place any significant constraints to the proposed development unless any planned excavations for underground utilities would extend 20 to 30 feet below the existing grades. This is considered unlikely and may be further evaluated once improvement plans are made available.

Perched groundwater conditions along fill/old paralic deposit contacts, and along zones of contrasting permeabilities, may not be precluded from occurring in the future due to site irrigation, increased precipitation, poor drainage conditions, or damaged underground utilities, and should be anticipated. Should perched groundwater conditions develop, this office could assess the affected area(s) and provide the appropriate recommendations to mitigate the observed groundwater conditions

ROCK HARDNESS/EXCAVATION DIFFICULTY

As previously indicated, a dense well-cemented hardpan was encountered within the old paralic deposits. Based on our experience, it is our opinion that the onsite earth materials will be generally rippable assuming the use of a Caterpillar D-9L bulldozer or equivalent. However, localized areas requiring heavy ripping cannot be entirely precluded. This hardpan may present moderate to extreme difficulty during planned excavations for foundation and underground utility trenching, especially if lightweight excavation equipment (i.e., backhoes and mini-excavators) are used. Thus, the client should consider overexcavating building pads, underground utility corridors, and street areas as discussed later in this report. The use of rock breaker attachments (i.e., hoe rams) may be necessary. Excavation equipment should be properly sized and powered for the required excavation task. If further information regarding rock hardness is necessary, this office can perform seismic refraction surveys to evaluate excavation feasibility.

GEOLOGIC HAZARDS EVALUATION

Mass Wasting/Landslide Susceptibility

Mass wasting refers to the various processes by which earth materials are moved down slope in response to the force of gravity. Examples of these processes include slope creep, surficial failures, and deep-seated landslides. Creep is the slowest form of mass wasting and generally involves the outer 5 to 10 feet of a slope surface. During heavy rains, such as those in El Niño years, creep-affected materials may become saturated, resulting in a more rapid form of downslope movement (i.e., landslides and/or surficial failures).

According to regional landslide susceptibility mapping by Tan and Giffen (1995), the site is located within landslide susceptibility Subareas 2 which is characterized as being "marginally susceptible" to landsliding. No evidence suggestive of deep-seated landsliding such as sheared clay seams, deep continuous fractures, failure scarps, or hummocky terrain was observed during our 2004 field studies. In addition, our review of 1953 stereoscopic aerial photographs (United States Department of Agriculture [USDA], 1953) did not indicate the presence of geomorphic features commonly associated with landslides (i.e., scarps and lobate debris fields). Given the above evidence, it is our opinion that the subject site has low susceptibility to deep-seated landslides. It should be noted that the onsite earth materials are erosive. Thus, properly designed surface drainage controls are considered necessary from a geotechnical perspective.

FAULTING AND REGIONAL SEISMICITY

Regional Faults

Our review indicates that there are no known active faults crossing the project area and the site is not within an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). However, the site is situated in a region subject to periodic earthquakes along active faults. According to Blake (2000a), the offshore segment of the Newport-Inglewood fault is the closest known active fault to the site and should produce the strongest ground accelerations should the design earthquake occur. Blake (2000a) shows this fault at a distance of approximately 5.3 miles [8.5 kilometers]) from the site. Cao, et al. (2003) indicate the slip rate on the offshore segment of the Newport-Inglewood fault is 1.5 ($\pm$0.5) millimeters per year (mm/yr) and the fault is capable of a maximum magnitude 7.1 earthquake. The location of the offshore segment of the Newport-Inglewood fault and other major faults within 100 kilometers of the site are shown on the “California Fault Map” in Appendix C of GSI (2004 see compact disc in Appendix A of this report). The possibility of ground acceleration, or shaking at the site, may be considered as approximately similar to the southern California region as a whole.

Local Faulting

Although active faults lie within a few miles of the site, no active faults were observed to specifically transect the site during the field investigation. Regional geologic mapping by Kennedy and Tan (2005) indicates a buried or concealed fault transecting the southeasterly corner of the subject site. This fault is not shown to displacement sediments younger than approximately 220,000 years. In addition, during our review of stereoscopic aerial photographs (USDA, 1953), a moderate to strong photo-lineament was observed in the northeasterly trending canyon to the southwest of the subject site. The trend of this photo-lineament does not cross the subject site. Given the above, it is our opinion that the subject site is at low risk to onsite faulting.

Surface Rupture

Surface rupture is the displacement of the ground surface from active fault movement. Owing to the lack of known active faults crossing the site, the potential for the proposed development to be adversely affected by surface rupture from active fault movement is considered very low.

Seismicity

Deterministic Site Acceleration

During preparation of GSI (2004), the acceleration-attenuation relationships of Bozorgnia, Campbell, and Niazi (1999) and Campbell and Bozorgnia (1997 Revised) were incorporated into EQFAULT (Blake, 2000a). EQFAULT is a computer program developed by Thomas F. Blake (2000a), which performs deterministic seismic hazard analyses using digitized California faults as earthquake sources.

The program estimates the closest distance between each fault and a given site. If a fault is found to be within a user-selected radius, the program estimates peak horizontal ground acceleration that may occur at the site from an upper bound (formerly “maximum credible earthquake”), on that fault. Upper bound refers to the maximum expected ground acceleration produced from a given fault. Based on the EQFAULT program, the site would experience a peak horizontal ground acceleration on the order of 0.55 g to 6.4 g from an upper bound event on the offshore segment of the Newport-Inglewood fault. The computer printouts of pertinent portions of the EQFAULT program are included within Appendix C of GSI (2004 [refer to the compact disc included in Appendix A of this report].

Updated Historical Site Seismicity

Historical site seismicity was updated using the acceleration-attenuation relationship of Bozorgnia, Campbell, and Niazi (1999), and the computer program EQSEARCH (Blake, 2000b, updated to January, 2015). This program performs a search of the historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-kilometer radius, between the years 1800 through January, 2015. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have affected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 through January, 2015 was about 0.193 g. A historic earthquake epicenter map and a seismic recurrence curve are also estimated/generated from the historical data. Computer printouts of the updated historical site seismicity evaluation, generated from EQSEARCH are presented in Appendix B of this report.

Seismic Shaking Parameters

Based on the site conditions, the following table summarizes the site-specific design criteria obtained from the 2013 CBC (CBSC, 2013), Chapter 16 Structural Design, Section 1613, Earthquake Loads. The computer program “U.S. Seismic Design Maps, provided by the United States Geologic Survey (USGS, 2014) was utilized for design (http://geohazards.usgs.gov/designmaps/us/application.php). The short spectral response utilizes a period of 0.2 seconds.

2013 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	**VALUE**	**2013 CBC AND/OR REFERENCE**
Site Class	D	Section 1613.3.2/ASCE 7-10 (Chapter 20)
Spectral Response - (0.2 sec), S_s	1.137 g	Figure 1613.3.1(1)
Spectral Response - (1 sec), S_1	0.437 g	Figure 1613.3.1(2)

2013 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	**VALUE**	**2013 CBC AND/OR REFERENCE**
Site Coefficient, F_a	1.045	Table 1613.3.3(1)
Site Coefficient, F_v	1.563	Table1613.3.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.189 g	Section 1613.3.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.683 g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.792 g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.456 g	Section 1613.3.4 (Eqn 16-40)
Seismic Design Category	D	Section 1613.3.5/ASCE 7-10 (Table 11.6-1 or 11.6-2)
PGA_M	0.469 g	ASCE 7-10 (Eqn 11.8.1)

GENERAL SEISMIC PARAMETERS	
PARAMETER	**VALUE**
Distance to Seismic Source (Newport Inglewood [offshore segment])	5.3 mi (8.5 km)[1]
Upper Bound Earthquake (Newport Inglewood [offshore segment])	$M_w = 7.1$[2]
[1] - Blake (2000a) [2] - Cao, et al. (2003)	

Conformance to the criteria above for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to eliminate all damage, since such design may be economically prohibitive. Cumulative effects of seismic events are not addressed in the 2013 CBC (CBSC, 2013) and regular maintenance and repair following locally significant seismic events (i.e., M_w5.5) will likely be necessary, as is the case in all of southern California.

SECONDARY SEISMIC HAZARDS

Liquefaction/Lateral Spreading

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 vertical deformation, lateral movement, lurching, sliding, and as a result of seismic loading, volumetric strain and manifestation in surface settlement of loose sediments, sand boils and other damaging lateral deformations. This phenomenon occurs only below the water table, but after liquefaction has developed, it can propagate upward into overlying non-saturated soil as excess pore water dissipates.

One of the primary factors controlling the potential for liquefaction is depth to groundwater. Typically, liquefaction has a relatively low potential at depths greater than 50 feet and is unlikely and/or will produce vertical strains well below 1 percent for depths below 60 feet when relative densities are 40 to 60 percent and effective overburden pressures are two or more atmospheres (i.e., 4,232 pounds per square foot [Seed, 2005]).

The condition of liquefaction has two principal effects. One is the consolidation of loose sediments with resultant settlement of the ground surface. The other effect is lateral sliding. Significant permanent lateral movement generally occurs only when there is significant differential loading, such as fill or natural ground slopes within susceptible materials. These conditions do not exist at the site.

Liquefaction susceptibility is related to numerous factors and the following five conditions should be concurrently present for liquefaction to occur: 1) sediments must be relatively young in age and not have developed a large amount of cementation; 2) sediments must generally consist of medium- to fine-grained, relatively cohesionless sands; 3) the sediments must have low relative density; 4) free groundwater must be present in the sediment; and 5) the site must experience a seismic event of a sufficient duration and magnitude, to induce straining of soil particles. Only about one to perhaps two of these five necessary concurrent conditions have the potential to affect the site.

Seismic Densification

Seismic densification is a phenomenon that typically occurs in low relative density granular soils (i.e., United States Soil Classification System [USCS] soil types SP, SM, SW, and SC) that are above the groundwater table. These unsaturated granular soils are susceptible if left in the original density (unmitigated), and are generally dry of the optimum moisture content (as defined by the ASTM D 1557). During seismic-induced ground shaking, these natural or artificial soils deform under loading and volumetrically strain, potentially resulting in ground surface settlements. Some densification of the adjoining un-mitigated properties may influence improvements at the perimeter of the site. Special setbacks and/or deepened foundations may be utilized if significant structures/improvements are placed close to the perimeter of the site. Our evaluation assumed that the current offsite conditions will not be significantly modified by future grading at the time of the design earthquake, which is a reasonably conservative assumption.

Summary

It is the opinion of GSI that the susceptibility of the site to experience damaging deformations from seismically-induced liquefaction and densification is relatively low owing to the dense, nature of the old paralic deposits that underlie the site in the near-surface. In addition, the recommendations for remedial earthwork and foundations would further reduce any significant liquefaction/densification potential. Some seismic densification of the adjoining un-mitigated site(s) may adversely influence planned improvements at the perimeter of the site. However, given the remedial earthwork and foundation recommendations provided herein, the potential for the planned buildings to be affected by significant seismic densification or liquefaction of offsite soils may be considered low.

Other Geologic/Secondary Seismic Hazards

The following list includes other geologic/seismic related hazards that have been considered during our evaluation of the site. The hazards listed are considered negligible and/or mitigated as a result of site location, soil characteristics, and typical site development procedures:

- Subsidence
- Ground Lurching or Shallow Ground Rupture
- Tsunami
- Seiche

SLOPE STABILITY

BEI (2015) indicates that planned cut and fill slopes will be constructed to heights on the order of 1½ feet and 1 foot, respectively. Such slopes are considered grossly and surficial stable assuming normal rainfall and irrigation, and provided that the slopes are maintained throughout the life of the development.

Site soils may be erosive. Precautionary measures, such a jute matting, hay bales, vegetative cover, berms, etc., are recommended to enhance the surficial stability of slopes until permanent vegetative covering is established. Vegetation should consist of deep-rooted flora that is capable of surviving the prevailing semi-arid climate with little to no water.

PREVIOUS LABORATORY TESTING

Laboratory tests were previously performed on representative samples of site earth materials collected during the subsurface exploration performed in preparation of GSI (2004). The testing procedures and results are summarized below.

Classification

Soils were visually classified with respect to the Unified Soil Classification System (U.S.C.S.) in general accordance with ASTM D 2487 and D 2488. The soil classifications of the onsite soils are provided on the Test Pit Logs in Appendix B of GSI (2004 [refer to the compact disc in Appendix A of this report]).

Laboratory Standard

The maximum density and optimum moisture content was evaluated for the major soil type encountered in the test pits completed in 2004. Testing was performed in general accordance with ASTM D 1557. The moisture-density relationships obtained for these soils are shown on the following table:

SAMPLE LOCATION AND DEPTH (FT)	SOIL TYPE	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
TP-1 @ 1-2, TP-1 @ 2-3 and TP-3 @ 2-3 (Composite Sample)	Brown, SANDY CLAY	128.5	9.5

Expansion Index

Representative samples of near-surface site soils collected during our 2004 subsurface exploration program was evaluated for expansion potential. Expansion Index (E.I.) testing and expansion potential classification was performed in general accordance with ASTM Standard D 4829, the results of the expansion testing are presented in the following table.

SAMPLE LOCATION AND DEPTH (FT)	EXPANSION INDEX	EXPANSION POTENTIAL
TP-1 @ 1-2, TP-1 @ 2-3, and TP-3 @ 2-3 (Composite Sample)	76	Medium
TP-3 @ 2-3	126	High

Direct Shear Test

Shear testing was performed on a representative (composite), “remolded” sample of site soil collected during our 2004 subsurface exploration program in general accordance with ASTM Test Method D-3080 in a Direct Shear Machine of the strain control type. The shear test results are presented as follows and are provided in Appendix D of GSI (2004 [refer to compact disc in Appendix A of this report]).

TEST PIT AND DEPTH (FT)	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
TP-1 @ 1-2, TP-1 @ 2-3, and TP-3 @ 2-3 (Composite Sample)	372	23	379	23

Atterberg Limits

Atterberg Limits Testing was performed on representative fine-grained soil samples collected during our 2004 subsurface exploration program to evaluate their liquid limit, plastic limit, and plasticity index (P.I.) in general accordance with ASTM D 4318-4318. The test results are presented below and in Appendix D of GSI (2004 [refer to compact disc in Appendix A of this report]). Testing indicates that the soil sample exhibits medium plastic behavior.

SAMPLE LOCATION AND DEPTH (FT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
TP-1 @ 1-2, TP-1 @ 2-3, and TP-3 @ 2-3 (Composite Sample)	48	20	28
TP-3 @ 2-3	62	22	40

Consolidation Testing

Consolidation testing was performed on a relatively undisturbed soil sample collected from Test Pit TP-1 at an approximate depth of 2 feet during our 2004 subsurface exploration program. Testing was performed in general accordance with ASTM Test Method D-2435-90. The consolidation test result is presented in Appendix D of GSI (2004 [refer to compact disc in Appendix A of this report]). The testing indicated that the sample compressed approximately 0.28 percent under a 1,000 pound per square foot (psf) load prior to inundation. Following innundation, the sample swelled approximately 2 percent under a 1,000 psf load. The sample then exhibited approximately 3.8 percent of compression under a 10,000 psf load.

Saturated Resistivity, pH, and Soluble Sulfates, and Chlorides

GSI conducted sampling and testing of the onsite earth materials for general soil corrosivity and soluble sulfates, and chlorides in preparation of GSI (2004). The testing included evaluation of soil pH, soluble sulfates, chlorides, and saturated resistivity. Test results are presented in the following table:

SAMPLE LOCATION AND DEPTH (FT)	pH	SATURATED RESISTIVITY (ohm-cm)	SOLUBLE SULFATES (% by weight)	SOLUBLE CHLORIDES (ppm)
TP-1 @ 1-2, TP-1 @ 2-3 and TP-3 @ 2-3 (Composite Sample)	7.7	1,600	0.0031	Non-detectable

Corrosion Summary

Laboratory testing indicates that tested samples of the onsite soils are: mildly alkaline with respect to soil acidity/alkalinity; corrosive to exposed, buried metals when saturated; present a negligible ("not applicable" per American Concrete Institute [ACI] 318-11) sulfate exposure to concrete; and non-detectable concentrations of soluble chlorides. It should be noted that GSI does not consult in the field of corrosion engineering. Thus, the client and project architect should agree on the level of corrosion protection required for the project and seek consultation from a qualified corrosion consultant, as warranted.

Resistance Value (R-value) Testing

An evaluation of the R-values for representative samples of subgrade materials within Dixie and Grace Streets was previously performed during the preparation of GSI (2007). Testing was performed in general accordance with the latest revisions to the Department of Transportation, State of California, Material & Research Test Method No. 301. Test results obtained from the representative samples of subgrade materials indicated R-values ranging between 9 and 13. R-value test results are included in Appendix C of GSI (2007 [refer to compact disc in Appendix A of this report]).

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our field exploration, laboratory testing, and geotechnical engineering analysis, it is our opinion that the subject site is suitable for the proposed convalescent development from a geotechnical engineering and geologic viewpoint, provided that the recommendations presented in the following sections are incorporated into the design and construction phases of site development. The primary geotechnical concerns with respect to the proposed development and improvements are:

- Earth materials characteristics and depth to competent bearing material.
- On-going expansion and corrosion potential of site soils.
- Rock hardness/excavation feasibility
- Erosiveness of site earth materials.
- Potential for perched water during and following site development.
- Perimeter conditions and planned improvements near the property boundary.
- Monitoring of adjacent property and temporary slopes during planned excavations near the property lines.
- The control of surface and subsurface water.
- Temporary and permanent slope stability, and the possible need for shored excavations along the westerly, easterly, and northerly property lines.
- The possible need to design planned retaining walls along the northerly, easterly, and westerly property lines for surcharge from adjacent structures and vehicular traffic.
- Regional seismic activity.

The recommendations presented herein consider these as well as other aspects of the site. The engineering analyses performed concerning site preparation and the recommendations presented herein have been completed using the information provided and obtained during our field work.

In the event that any significant changes are made to proposed site development, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the recommendations of this report verified or modified in writing by this office. Foundation design parameters are considered preliminary until the foundation design, layout, and structural loads are provided to this office for review. The most significant findings of our investigation as summarized below.

1. Quaternary-age colluvium, the Quaternary-age paleosol, and weathered portions of the Quaternary-age old paralic deposits are considered unsuitable for the support of the planned settlement-sensitive improvements (i.e., buildings, walls, underground utilities, pavements, etc.) and new planned fills. Unweathered old paralic deposits are considered suitable bearing materials. Based on the available subsurface data, the depth of remedial grading excavations to encounter suitable bearing materials is anticipated to range between approximately ½ foot and 4½ feet below the existing grades. However, they may occur deeper, locally. This should be considered during project planning and construction.

2. Expansion index (E.I.) testing, performed on samples of the onsite soil in preparation of GSI (2004), indicates some of the onsite soils have expansion indices ranging between 76 and 126. This corresponds to medium and high expansion potentials, respectively. Previous Atterberg limits testing on these samples indicates plasticity indices (P.I.s) ranging between 28 and 40. Based on these test results, the onsite soils meet the criteria of detrimentally expansive soils, as indicated in Section 1803.5.3. of the 2013 CBC (CBSC, 2013). In order to comply

with 2013 CBC requirements for the mitigation of expansive soils, the proposed structures will need specific foundation and slab-on-grade design that will tolerate the shrink/swell effects of expansive soils (see Section 1808.6.2 of the 2013 CBC). Alternatively, expansive soils within the influence of the proposed structures may be removed and replaced with very low expansive soils (E.I. less than 21) with a plasticity index (P.I.) less than 15 (see Section 1808.6.3 of the 2013 CBC). These mitigative scenarios are further discussed herein.

3. Previous laboratory testing, performed to evaluate the corrosiveness of a representative sample of the onsite soils in preparation of GSI (2004), indicates that soil is mildly alkaline with respect to soil acidity/alkalinity; corrosive to exposed, buried metals when saturated; presents a negligible ("not applicable" per American Concrete Institute [ACI] 318-11) sulfate exposure to concrete; and non-detectable concentrations of soluble chlorides. It should be noted that GSI does not consult in the field of corrosion engineering. Thus, the client and project architect should agree on the level of corrosion protection required for the project and seek consultation from a qualified corrosion consultant, as warranted.

4. During our subsurface exploration, performed in preparation of GSI (2004), a dense, cemented hardpan was encountered within some of our test pits. This hardpan may create difficult excavations with lightweight excavation equipment such as backhoes and mini-excavators, and possibly heavier excavators. However, based on our experience, it should be rippable with a Caterpillar D-9L bulldozer. Given the potential for difficult excavation, the client may considered overexcavating street/driveway areas and underground utility corridors to at least 1 foot below the lowest underground utility invert and replacing the overexcavated materials with engineered fill during grading to help facilitate underground utility construction.

5. The onsite soils are considered erosive. Therefore, engineered surface drainage controls are recommended. Vegetative and surface coverings (i.e., pavements and hardscape) will also provide protection from surficial erosion. Temporary erosion control measures should be provided until vegetation covering is well established and planned pavements are constructed.

6. No evidence of a high regional groundwater table nor perched water was observed during the subsurface exploration performed in preparation of GSI (2004). However, due to the nature of site earth materials, there is a potential for perched water to occur both during and following site development. This potential should be disclosed to all interested/affected parties. Should perched water conditions be encountered, this office could provide recommendations for mitigation. Typical mitigation includes subdrainage system, cut-off barriers, etc.

7. Removal and recompaction of potentially compressible soils below a 1:1 (h:v) projection down from the bottom outside of planned settlement-sensitive improvements and new planned fills along the perimeter of the site will be limited

due to boundary restrictions. As such, any settlement-sensitive improvement located above a 1:1 (h:v) projection from the bottom outboard edge of the remedial grading excavation at the property line would require deepened foundations below this plane, additional reinforcement, or would retain some potential for distress and therefore, a reduced serviceable life. On a preliminary basis, any planned settlement-sensitive improvements located within approximately ½-foot and 4½ feet from the property lines would require deepened foundations or additional reinforcement by means of ground improvement or specific structural design. This should be considered during project design. Footings for the planned retaining walls and privacy walls along the northerly, easterly, and westerly property margins will likely require deepened footings that extend into unweathered old paralic deposits since boundary restrictions could constrain remedial grading from occurring below a 1:1 (h:v) projection down from the bottom edge of the retaining wall footings.

8. Excavations for planned retaining wall construction near the northerly, easterly, and westerly property lines may require shoring for the protection of offsite property. It is recommended to perform surveys of offsite improvements prior to completing planned excavations and monitoring of these excavations during construction. There may be insufficient room to construct geogrid-reinforced retaining walls.

9. Planned retaining walls and privacy walls located below a 1:1 (horizontal:vertical [h:v]) plane down from the bottom outboard edge of offsite improvements including walls and structures should be designed for surcharge. In addition, traffic surcharge should be considered in the design of planned retaining walls if vehicular/traffic loads will occur within the aforementioned plane.

10. On a preliminary basis, temporary slopes should be constructed in accordance with CAL-OSHA guidelines for Type "B" soils, provided water or seepage or running sands are not present. All temporary slopes should be evaluated by the geotechnical consultant, prior to worker entry. Should adverse conditions be identified, the slope may need to be laid back to a flatter gradient or require the use of shoring.

11. Given the site's location within a seismically active region, the planned improvements may be subject to moderate to strong ground shaking over their design life. From a life/safety standpoint, the seismicity-acceleration values provided herein should be considered during the design and construction of the proposed development.

12. General Earthwork, Grading Guidelines, and Preliminary Criteria are provided at the end of this report as Appendix C. Specific recommendations are provided below.

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

All earthwork should conform to the guidelines presented in the 2013 CBC (CBSC, 2013), the requirements of the City of Oceanside, and the General Earthwork and Grading Guidelines presented in Appendix C, except where specifically superceded in the text of this report. Prior to earthwork, a GSI representative should be present at the preconstruction meeting to provide additional earthwork guidelines, if needed, and review the earthwork schedule. This office should be notified in advance of any fill placement, supplemental regrading of the site, or backfilling underground utility trenches and retaining walls after rough earthwork has been completed. This includes grading for driveway approaches, driveways, and exterior hardscape.

During earthwork construction, all site preparation and the general grading procedures of the contractor should be observed and the fill selectively tested by a representative(s) of GSI. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and, if warranted, modified and/or additional recommendations will be offered. All applicable requirements of local and national construction and general industry safety orders, the Occupational Safety and Health Act (OSHA), and the Construction Safety Act should be met. It is the onsite general contractor's and individual subcontractors' responsibility to provide a safe working environment for our field staff who are onsite. GSI does not consult in the area of safety engineering. It is also the responsibility of the contractor to provide protection of their work product. Surface drainage should be directed away from open excavations.

Site Preparation

All existing improvements, vegetation, stockpiled materials, and deleterious debris should be removed from the site, prior to the start of construction, if they are located in areas of proposed earthwork.

Any remaining cavities resulting from the removal of any existing improvements should be observed by the geotechnical consultant. Mitigation of cavities would likely include removing any potentially compressible soils to expose unweathered old paralic deposits and then backfilling the excavation with a controlled engineered fill or soils that have been moisture conditioned to optimum moisture content and compacted to at least 90 percent of the laboratory standard (ASTM D 1557).

It is possible that an onsite sewage disposal system may be present. Should such structures be encountered during earthwork, this office should be contacted to provide recommendations for removal and disposal.

Remedial Excavation - Removal and Recompaction of Potentially Compressible Earth Materials

Potentially compressible undocumented fill, colluvium, and weathered old paralic deposits should be removed to expose unweathered old paralic deposits. Following remedial excavation, these soils should be cleaned of any vegetation and deleterious debris, moisture conditioned to at least the soil's optimum moisture content, and then be reused as engineered fill in accordance with the recommendations in the "Fill Placement"section below. Based on the available subsurface data, excavations necessary to remove unsuitable soils are anticipated to range between approximately ½-foot and 4½ feet below existing grades. The potential to encounter thicker sections of unsuitable soils that require deeper remedial grading excavations, than stated above, cannot be precluded and should be anticipated. Potentially compressible soils should be removed below a 1:1 (h:v) projection down from the bottom, outboard edge of any settlement-sensitive improvement or limits of planned fill unless constrained by property lines or onsite improvements that need to remain in serviceable use both during and following site development. Remedial grading excavations should be observed by the geotechnical consultant prior to scarification and fill placement. Once observed and approved, the bottom of the remedial grading excavation should be scarified at least 6 to 8 inches, moisture conditioned to at least the soil's optimum moisture content, and then be recompacted in accordance with the recommendations in the "Fill Placement"section below.

Cut/Fill Transitions and Overexcavation

Foundations should be supported by a uniform thickness, engineered fill blanket. This will require that any unweathered old paralic deposits exposed within 42 inches of pad grade or 24 inches below the lowest foundation element be overexcavated and replaced with engineered fill. The bottoms of overexcavations should be sloped toward the streets, driveways, or approved drainage facility, and be observed by the geotechnical consultant prior to scarification and backfill. Once approved by the geotechnical consultant, overexcavation bottoms should be scarified at least 6 to 8 inches, moisture conditioned to at least the soil's optimum moisture content and then be recompacted to at least 90 percent of the laboratory standard (ASTM D 1557). The maximum to minimum fill thickness in the building pad areas should not exceed a ratio of 3:1 (maximum:minimum). This engineered fill cap is not intended to mitigate expansive soils. Earthwork mitigation recommendations for expansive soils is provided in the following section.

Earthwork Mitigation of Expansive Soils

The following recommendations are intended to comply with the requirements of Section 1808.6.3 of the 2013 CBC if specific structural design for the mitigation of expansive soils (per Section 1808.6.2) will not be incorporated into the project. This mitigative work would include the overexcavation of expansive soils/bedrock to a sufficient depth such that the weighted plasticity index of replacement fills within the influence of the foundation/slab-on-grade floor system will have a P.I. less than 15. The following preliminary table provides recommended overexcavation depths for the use of very low expansive (E.I. 0 to 20) replacement fills with certain assumed plasticity indices. Our evaluation assumes that the soils within the upper 15 feet of pad grade, prior to mitigation, have a P.I. of 40. Additional lot-specific exploration and testing should be performed prior to or during grading to further evaluate the P.I. of soils within 15 feet of pad grade and in order to refine the recommended overexcavation depths provided herein. The lateral extent of the overexcavation, outside the building footprint, should be equal to the depth of the overexcavation.

PLASTICITY INDEX (P.I.) OF REPLACEMENT FILLS	OVEREXCAVATION DEPTH BELOW PAD GRADE (FT)
0	7
5	8½
10	11
14	15

It should be noted that import will likely be necessary for the replacement fill. Additional studies could be performed to evaluate if mining of the granular old paralic deposits for use in expansive soil mitigation is feasible.

Perimeter Conditions

It should be noted that the 2013 CBC (CBSC, 2013) indicates that removals of unsuitable soils be performed across all areas to be graded, under the purview of the grading permit (if required), not just within the influence of the planned structures. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas. This zone would be approximately equal to the depth of removals, if removals cannot be performed onsite or offsite. In general, any planned improvement located above a 1:1 (h:v) projection up from the bottom, outboard edge of the remedial grading excavation at the property boundaries or adjacent to existing improvements that need to remain and serviceable use both during and following site development would be affected by perimeter conditions. On a preliminary basis, any planned settlement-sensitive improvements located within approximately ½ foot and 4½ feet from the property lines would require deepened foundations or additional reinforcement by means of ground improvement or specific structural design, for the perimeter conditions discussed above. Otherwise, these improvements may be subject to distress and a reduced serviceable life span. This would require proper disclosure to all interested/affected parties should this condition exist at the conclusion of grading. If remedial grading cannot be performed below a 1:1 (h:v) projection down from the bottom outboard edges of the planned retaining wall and privacy wall footings along the northerly, westerly, and easterly property lines, the footings would need to extend into unweathered old paralic deposits for vertical and lateral bearing support.

Fill Placement

Following scarification of the bottom of the remedial grading excavations, the reused onsite soils and import (if necessary) should be placed in ±6- to ±8-inch lifts, cleaned of vegetation and debris, moisture conditioned to at least the soil's optimum moisture content, and compacted to achieve a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557). Fill materials should not contain rock constituents greater than 12 inches in any dimension. Keying and benching is recommended during the placement of fill on surfaces with gradients steeper than 5:1 (h:v).

Import Soils

If import fill is necessary, a sample of the soil import should be evaluated by this office prior to importing, in order to assure compatibility with the onsite soils and the recommendations presented in this report. If non-manufactured materials are used, environmental documentation for the export site should be provided for GSI review. At least five business days of lead time should be allowed by builders or contractors for proposed import submittals. This lead time will allow for environmental document review, particle size analysis, laboratory standard, expansion testing, and blended import/native characteristics as deemed necessary. If earthwork mitigation of expansive soils is not incorporated into the grading of the site import soils may have an expansion index of 90 or less. Import for the use of expansive soil mitigation should be non-detrimentally expansive (i.e., E.I. less than 21 and P.I. less than 15). The use of subdrains at the bottom of the fill cap may be necessary, and may be subsequently recommended based on compatibility with onsite soils. Import soils for retaining wall backfill should consist of select backfill with an E.I. less than 20, a P.I. of 6 or less, and 15 percent or less passing the No. 200 sieve.

Graded Slope Construction

Graded cut slopes should be observed by a representative of this firm during grading to evaluate the presence of adverse geologic structures (daylighted joint and/or fracture planes), undocumented fills, highly weathered and fractured old paralic deposits, or other geologic conditions that could affect cut slope stability. Although not anticipated, should adverse conditions be exposed, mitigation measures would be recommended. Such mitigation measures may include but not necessarily be limited to: inclining cut slopes to flatter gradients, stabilization fills, etc. A detail showing recommended stabilization fills is provided in Appendix C.

Graded fill slopes should be properly keyed and benched, and be compacted to at least 90 percent relative compaction throughout, including the slope face. Compaction at the slope face may be achieved by either overbuilding and trimming back fill slopes or back-rolling fill slopes with compaction equipment every 4 vertical feet. Fill materials used in fill slope construction should have a minimum cohesion © of 200 and a minimum friction angle (ϕ) of 29 degrees.

Graded slopes should receive vegetative covering immediately following construction. In the interim between construction and the establishment of landscape cover, the graded slopes should receive City approved erosion control devices.

Temporary Slopes

Temporary slopes for excavations greater than 4 feet, but less than 20 feet in overall height should conform to CAL-OSHA and/or OSHA requirements for Type "B" soils, provided water or seepage and/or running sands are not present. Temporary slopes, up to a maximum height of $\pm$20 feet, may be excavated at a 1:1 (h:v) gradient, or flatter, provided groundwater and/or running sands are not exposed. Construction materials, soil stockpiles, and heavy equipment should not be stored and/or operated within 'H' of any temporary slope where 'H' equals the height of the temporary slope. All temporary slopes should be observed by a licensed engineering geologist and/or geotechnical engineer prior to worker entry into the excavation. Based on the exposed field conditions, inclining temporary slopes to flatter gradients or the use of shoring may be necessary if adverse conditions are observed. If adverse conditions are exposed or if temporary slopes conflict with property boundaries or existing improvements to remain following development, shoring or alternating slot excavations may be necessary. The need for shoring or alternating slot excavations could be further evaluated during the grading plan review stage.

Excavation Observation and Monitoring (All Excavations)

When excavations are made adjacent to an existing improvement (i.e., utility, wall, road, building, etc.) there is a risk of some damage even if a well designed system of excavation is planned and executed. We recommend, therefore, that a systematic program of observations be made before, during, and after construction to determine the effects (if any) of construction on existing improvements.

We believe that this is necessary for two reasons: First, if excessive movements (i.e., more than ½ inch) are detected early enough, remedial measures can be taken which could possibly prevent serious damage to existing improvements. Second, the responsibility for damage to the existing improvement can be determined more equitably if the cause and extent of the damage can be determined more precisely.

Monitoring should include the measurement of any horizontal and vertical movements of the existing structures/improvements. Locations and type of the monitoring devices should be selected prior to the start of construction. The program of monitoring should be agreed upon between the project team, the site surveyor and the Geotechnical Engineer-of-Record, prior to excavation.

Reference points on existing walls, buildings, and other settlement-sensitive improvements. These points should be placed as low as possible on the wall and building adjacent to the excavation. Exact locations may be dictated by critical points, such as bearing walls or columns for buildings; and surface points on roadways or curbs near the top of the excavation.

For a survey monitoring system, an accuracy of a least 0.01 foot should be required. Reference points should be installed and read initially prior to excavation. The readings should continue until all construction below ground has been completed and the permanent backfill has been brought to final grade.

The frequency of readings will depend upon the results of previous readings and the rate of construction. Weekly readings could be assumed throughout the duration of construction with daily readings during rapid excavation near the bottom of the excavation. The reading should be plotted by the Surveyor and then reviewed by the Geotechnical Engineer. In addition to the monitoring system, it would be prudent for the Geotechnical Engineer and the Contractor to make a complete inspection of the existing structures both before and after construction. The inspection should be directed toward detecting any signs of damage, particularly those caused by settlement. Notes should be made and pictures should be taken where necessary.

Observation

It is recommended that all excavations be observed by the Geologist and/or Geotechnical Engineer. Any fill which is placed should be approved, tested, and verified if used for engineered purposes. Should the observation reveal any unforseen hazard, the Geologist or Geotechnical Engineer will recommend treatment. Please inform GSI at least 24 hours prior to any required site observation.

PRELIMINARY RECOMMENDATIONS - FOUNDATIONS

General

Preliminary recommendations for foundation design and construction are provided in the following sections. These preliminary recommendations have been developed from our understanding of the currently planned site development, site observations, subsurface exploration, laboratory testing, and engineering analyses. Foundation design should be re-evaluated at the conclusion of site grading/remedial earthwork for the as-graded soil conditions. Although not anticipated, revisions to these recommendations may be necessary. In the event that the information concerning the proposed development plan is not correct, or any changes in the design, lot layouts, or loading conditions of the proposed buildings are made, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report are modified or approved in writing by this office.

The information and recommendations presented in this section are not meant to supercede design by the project structural engineer or civil engineer specializing in structural design. Upon request, GSI could provide additional input/consultation regarding soil parameters, as related to foundation design.

In the following subsections, GSI provides preliminary design and construction recommendations for conventional foundation and slab-on-grade floor systems underlain and laterally juxtaposed by non-detrimentally expansive soils (E.I. ≤ 20 and P.I. < 15) and post-tensioned, and mat foundation systems within the influence of detrimentally expansive soils (E.I. > 20 and P.I. > 14). GSI's recommended earthwork mitigation for the treatment of expansive soils will be necessary for the use of conventional foundation and slab-on-grade floor systems.

Preliminary Foundation Design

1. The foundation systems should be designed and constructed in accordance with guidelines presented in the 2013 CBC.

2. An allowable bearing value of 2,000 pounds per square foot (psf) may be used in the design of continuous spread footings that maintain a minimum width of 12 inches and a minimum depth of 12 inches below the lowest adjacent grade. This value may also be used for the design of isolated spread footings that maintain a minimum dimension of 24 square inches and a minimum depth of 24 inches below the lowest adjacent grade. Foundation embedment depth excludes concrete slabs-on-grade, and/or slab underlayment. The allowable bearing value may be increased by 20 percent for each additional 12 inches in footing depth to a maximum value of 2,500 psf. The allowable bearing value may be increased by one-third when considering short duration seismic or wind loads. The above values may be used provided that the footings are founded into engineered fill overlying dense old paralic deposits that have been approved by the geotechnical consultant.

3. For foundations deriving passive resistance from approved non-detrimentally expansive engineered fill, a passive earth pressure may be computed as an equivalent fluid having a density of 250 pcf, with a maximum earth pressure of 2,500 psf. The upper 6 inches of passive pressure should be neglected if not confined by slabs or pavement.

4. For lateral sliding resistance, a 0.35 coefficient of friction may be utilized for a concrete to soil contact when multiplied by the dead load. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

5. All footing setbacks from slopes should comply with Figure 1808.7.1 of the 2013 CBC. GSI recommends a minimum horizontal setback distance of 10 feet as measured from the bottom, outboard edge of the footing to the slope face. Foundations setbacks from slopes on the westerly side of APN 149-350-50 should be increased to 15 feet. Foundations should also extend below a 1:1 (h:v) projection up from the bottom outside edge of remedial grading excavations.

6. Footings for structures adjacent to retaining walls should be deepened so as to extend below a 1:1 (h:v) projection up from the heel of the wall.

7. Provided that the earthwork and foundation recommendations in this report are adhered, foundations bearing on approved non-detrimentally expansive, engineered fil overlying dense old paralic deposits should be minimally designed to accommodate a total static settlement of 2 inches and a differential static settlement of 1-inch over a 40-foot horizontal span (angular distortion = 1/480), and up to ½ inch of seismic differential settlement over a 40-foot horizontal span (seismic angular distortion = 1/960).

PRELIMINARY CONVENTIONAL FOUNDATION AND SLAB-ON-GRADE CONSTRUCTION RECOMMENDATIONS

The following recommendations are for building foundations and slab-on-grade floor systems underlain and laterally juxtaposed by non-detrimentally expansive soils (i.e., E.I. < 21 and P.I. < 15), where the weighted plasticity index within the upper 15 feet of foundation soils is less than 15. The structural engineer's recommendations may more onerous, based on actual floor loads.

1. Exterior and interior footings should be founded into approved engineered fill at a minimum depth of 12 or 18 inches below the lowest adjacent grade for a one- or two-story floor load, respectively. For one- and two-story floor loads, footing widths should be at least 12 and 15 inches, respectively. Isolated, exterior column and panel pads, or wall footings, should be at least 24 inches square, and founded at a minimum depth of 24 inches into approved engineered fill. All footings should be minimally reinforced with four No. 4 reinforcing bars, two placed near the top and two placed near the bottom of the footing. Depth of embedment does not include the slab or underlayment thickness, and is measured from the lowest adjacent grade.

2. All interior and exterior column footings, and perimeter wall footings, should be tied together via grade beams in at least one direction, for very low expansive soils. The grade beams should be at least 12 inches square in cross section, and should be provided with a minimum of one No.4 reinforcing bar near the top, and one No.4 reinforcing bar near the bottom of the grade beam. The base of the reinforced grade beams should be at the same elevation as the adjoining footings. A stepped grade beam, constructed per the structural engineer's specifications, may be necessary where the base of footings occur at different elevations.

3. A grade beam, reinforced as previously recommended and at least 12 inches square, should be provided across large (garage) entrances. The base of the reinforced grade beam should be at the same elevation as the adjoining footings. A stepped grade beam, constructed per the structural engineer's specifications, may be necessary where the base of footings occur at different elevations.

4. A minimum concrete slab-on-grade floor thickness of 4.5 inches is recommended. A maximum water to cement ratio of 0.5 is recommended for foundations and slab-on-grade floors.

5. Concrete slabs should be reinforced with a minimum of No. 3 reinforcement bars placed at 18 inches on center, in two horizontally perpendicular directions (i.e., long axis and short axis).

6. The actual thickness and steel reinforcement for concrete slab-on-grade floors should be determined by the project structural engineer, based on the anticipated loading conditions and building use. However, the slab thickness and steel reinforcement, recommended above, are considered minimum guidelines.

7. All slab reinforcement should be supported to ensure proper mid-slab height positioning during placement of the concrete. "Hooking" of reinforcement is not an acceptable method of positioning.

8. Slab subgrade pre-soaking is not required for non-detrimentally expansive soil conditions. However, the Client should consider pre-wetting the slab subgrade materials to at least the soil's optimum moisture content to a minimum depth of 12 inches, within 72 hours of the placement of the underlayment sand and vapor retarder.

9. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557), whether the soils are to be placed inside the foundation perimeter or in the yard/right-of-way areas. This material must not alter positive drainage patterns that direct drainage away from the structural areas and toward traffic areas or approved drainage facilities.

POST-TENSIONED FOUNDATION SYSTEMS

Post-tension foundations may be used to mitigate the damaging shrink/swell effects of expansive soils on the planned foundations and slab-on-grade floors. The post-tension foundation designer may elect to exceed the minimal recommendations, provided herein, to increase slab stiffness performance. Post-tension (PT) design may be either ribbed or mat-type. The latter is also referred to as uniform thickness foundation (UTF). The use of a UTF is an alternative to the traditional ribbed-type. The UTF offers a reduction in grade beams. That is to say a UTE typically uses a single perimeter grade beam and possible "shovel" footings, but has a thicker slab than the ribbed-type.

The information and recommendations presented in this section are not meant to supercede design by a registered structural engineer or civil engineer qualified to perform post-tensioned design. Post-tensioned foundations should be designed using sound engineering practice and be in accordance with local and 2013 CBC requirements. Upon request, GSI can provide additional data/consultation regarding soil parameters as related to post-tensioned foundation design. For the purpose of this study, GSI is providing recommended post-tension foundation design criteria for medium, high, and very high expansive soil conditions (E.I. = 51 to >130). Although not encountered during field work performed in preparation of GSI (2004), the possibility of exposing soils with a very high expansion potential during grading cannot entirely be precluded.

From a soil expansion/shrinkage standpoint, a common contributing factor to distress of structures using post-tensioned slabs is a "dishing" or "arching" of the slabs. This is caused by the fluctuation of moisture content in the soils below the perimeter of the slab primarily due to onsite and offsite irrigation practices, climatic and seasonal changes, and the presence of expansive soils. When the soil environment surrounding the exterior of the slab has a higher moisture content than the area beneath the slab, moisture tends to migrate inward, underneath the slab edges to a distance beyond the slab edges referred to as the moisture variation distance. When this migration of water occurs, the volume of the soils beneath the slab edges expands and causes the slab edges to lift in response. This is referred to as an edge-lift condition. Conversely, when the outside soil environment is drier, the moisture transmission regime is reversed and the soils underneath the slab edges lose their moisture and shrink. This process leads to dropping of the slab at the edges, which leads to what is commonly referred to as the center lift condition. A well-designed, post-tensioned slab having sufficient stiffness and rigidity provides a resistance to excessive bending that results from non-uniform swelling and shrinking slab subgrade soils, particularly within the moisture variation distance, near the slab edges. Other mitigation techniques typically used in conjunction with post-tensioned slabs consist of a combination of specific soil pre-saturation and the construction of a perimeter "cut-off" wall grade beam. Soil pre-saturation consists of moisture conditioning the slab subgrade soils prior to the post-tension slab construction. This effectively reduces soil moisture migration from the area located outside the building toward the soils underlying the post-tension slab. Perimeter cut-off walls are thickened edges of the concrete slab that impedes both outward and inward soil moisture migration.

Slab Subgrade Pre-Soaking

Pre-moistening of the slab subgrade soil is recommended for detrimentally expansive soil conditions. The moisture content of the subgrade soils should be equal to or greater than optimum moisture to a depth equivalent to the perimeter grade beam or cut-off wall depth in the slab areas (typically 18, 24, and 36 inches) for medium, high, and very high expansive soil conditions.

Pre-moistening and/or pre-soaking should be evaluated by the soils engineer 72 hours prior to vapor retarder placement. In summary:

EXPANSION POTENTIAL	PAD SOIL MOISTURE	CONSTRUCTION METHOD	SOIL MOISTURE RETENTION
Medium (E.I. = 51-90)	Upper 18 inches of pad soil moisture 2 percent over optimum (or 1.2 times)	Berm and flood or wetting and reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.
High (E.I. = 91-130)	Upper 24 inches of pad soil moisture 3 percent over optimum (or 1.3 times)	Berm and flood or wetting and reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.
Very High (E.I. > 130)	Upper 36 inches of pad soil moisture 3 percent over optimum (or 1.3 times)	Berm and flood or wetting and reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.

Perimeter Cut-Off Walls

Perimeter cut-off walls should be at least 18, 24, and 36 inches deep for medium, high, and very high expansive soil conditions, respectively. The cut-off walls may be integrated into the slab design or independent of the slab. The cut-off walls should be a minimum of 6 inches thick (wide). The bottom of the perimeter cut-off wall should be designed to resist tension, using cable or reinforcement per the structural engineer.

Post-Tensioned Foundation Design

The following recommendations for design of post-tensioned slabs have been prepared in general compliance with the requirements of the recent Post Tensioning Institute's (PTI's) publication titled "Design of Post-Tensioned Slabs on Ground, Third Edition" (PTI, 2004), together with it's subsequent addendums (PTI, 2008).

Soil Support Parameters

The recommendations for soil support parameters have been provided based on the typical soil index properties for soils that are very low to very high in expansion potential. The soil index properties are typically the upper bound values based on our experience and practice in the southern California area. Additional testing is recommended either during or following grading, and prior to foundation construction to further evaluate the soil conditions within the upper 7 to 15 feet of pad grade. The following table presents suggested minimum coefficients to be used in the Post-Tensioning Institute design method.

Thornthwaite Moisture Index	-20 inches/year
Correction Factor for Irrigation	20 inches/year
Depth to Constant Soil Suction	7 feet or overexcavation depth to bedrock
Constant soil Suction (pf)	3.6
Moisture Velocity	0.7 inches/month
Effective Plasticity Index (P.I.)*	15-45
* - The effective plasticity index should be evaluated for the upper 7 to 15 feet of foundation soils either during or following grading.	

Based on the above, the recommended soil support parameters are tabulated below:

DESIGN PARAMETERS	MEDIUM EXPANSION (E.I. = 51-90)	HIGH EXPANSION (E.I. = 91-130)	VERY HIGH EXPANSION (E.I. >130)
e_m center lift	8.7 feet	8.5 feet	7.5 feet
e_m edge lift	4.5 feet	4.0 feet	3.25 feet
y_m center lift	0.5 inches	0.66 inches	1.1 inches
y_m edge lift	1.3 inch	1.7 inches	2.5 inches
Bearing Value (1)	1,000 psf	1,000 psf	1,000 psf
Lateral Pressure	175 psf	150 psf	125 psf
Subgrade Modulus (k)	85 pci/inch	70 pci/inch	60 pci/inch
Minimum Perimeter Footing Embedment (2)	18 inches	24 inches	30 inches

(1) Internal bearing values within the perimeter of the post-tension slab may be increased to 1,500 psf for a minimum embedment of 12 inches, then by 20 percent for each additional foot of embedment to a maximum of 2,500 psf.
(2) As measured below the lowest adjacent compacted subgrade surface without landscape layer or sand underlayment.
Note: The use of open bottomed raised planters adjacent to foundations will require more onerous design parameters.

The parameters are considered minimums and may not be adequate to represent all expansive soils and site conditions such as adverse drainage and/or improper landscaping and maintenance. The above parameters are applicable provided the structure has positive drainage that is maintained away from the structure. In addition, no trees with significant root systems are to be planted within 15 feet of the perimeter of foundations. Therefore, it is important that information regarding drainage, site maintenance, trees, settlements, and effects of expansive soils be passed on to future all interested/affected parties. The values tabulated above may not be appropriate to account for possible differential settlement of the slab due to other factors, such as excessive settlements. If a stiffer slab is desired, alternative Post-Tensioning Institute ([PTI] third edition) parameters may be recommended. All exterior columns not supported by the post-tensioned foundation should be supported by 24 square inch isolated footings extending at least 24 inches below lowest adjacent grade into approved engineered fill. Exterior column footings should be tied to the post-tensioned foundation with 12 square inch, reinforced grade beams in at least two directions.

MAT FOUNDATIONS

In lieu of using a post-tensioned foundation to resist expansive soil effects, the Client may consider a mat foundation which uses steel bar reinforcement instead of post-tensioned cables. The structural engineer may supercede the following recommendations based on the planned building loads and use. WRI (Wire Reinforcement Institute) methodologies for design may be used. Mat foundations may be incorporate exterior and interior stiffening beams or a uniform thickness slab. Minimum mat embedment should be 12 inches below the lowest adjacent grade into approved engineered fill overlying dense old paralic deposits.

Mat Foundation Design

The design of mat foundations should incorporate the vertical modulus of subgrade reaction. This value is a unit value for a 1-foot square footing and should be reduced in accordance with the following equation when used with the design of larger foundations. This is assumes that the bearing soils will consist of engineered fills with an average relative compaction of 90 percent of the laboratory (ASTM D 1557), overlying dense old paralic deposits.

$$K_R = K_S\left[\frac{B+1}{2B}\right]^2$$

where: K_S = unit subgrade modulus
K_R = reduced subgrade modulus
B = foundation width (in feet)

The modulus of subgrade reaction (K_S) and effective plasticity index (PI) to be used in mat foundation design for various expansive soil conditions are presented in the following table. The effective plasticity index for the upper 7 to 15 feet of the foundation soils should be performed during or following grading. Lateral pressures for mat foundation design should conform to those previously provided in the “Post-Tensioned Foundation Systems” section of this report.

MEDIUM EXPANSION (E.I. = 51-90)	HIGH EXPANSION (E.I. = 91-130)	VERY HIGH EXPANSION (E.I. > 130)
K_S =85 pci/inch, PI = 25	K_S =70 pci/inch, PI = 35	K_S =60 pci/inch, PI = 45

All exterior columns not supported by the mat foundation should be supported by 24 square inch isolated footings extending at least 24 inches into approved engineered fill. Exterior column footings should be tied to the mat foundation with 12 square inch, reinforced grade beams in at least two directions.

SOIL MOISTURE TRANSMISSION CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through new concrete floor slabs, in light of typical floor coverings and improvements. Please note that slab moisture emission rates range from about 2 to 27 lbs/24 hours/1,000 square feet from a typical slab (Kanare, 2005), while floor covering manufacturers generally recommend about 3 lbs/24 hours as an upper limit. The recommendations in this section are not intended to preclude the transmission of water or vapor through the foundation or slabs. Foundation systems and slabs shall not allow water or water vapor to enter into the structure so as to cause damage to another building component or to limit the installation of the type of flooring materials typically used for the particular application (State of California, 2014). These recommendations may be exceeded or supplemented by a water “proofing” specialist, project architect, or structural consultant. Thus, the client will need to evaluate the following in light of a cost vs. benefit analysis (owner expectations and repairs/replacement), along with disclosure to all interested/affected parties. It should also be noted that vapor transmission will occur in new slab-on-grade floors as a result of chemical reactions taking place within the curing concrete. Vapor transmission through concrete floor slabs as a result of concrete curing has the potential to adversely affect sensitive floor coverings depending on the thickness of the concrete floor slab and the duration of time between the placement of concrete, and the floor covering. It is possible that a slab moisture sealant may be needed prior to the placement of sensitive floor coverings if a thick slab-on-grade floor is used and the time frame between concrete and floor covering placement is relatively short.

Considering the E.I. test results presented herein, and known soil conditions in the region, the anticipated typical water vapor transmission rates, floor coverings, and improvements (to be chosen by the Client and/or project architect) that can tolerate vapor transmission rates without significant distress, the following alternatives are provided:

- Concrete slabs, including the garage slab, should be thicker. The project structural engineer may require a thicker slab-on-grade to mitigate expansive soil conditions.

- Concrete slab underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2013 CBC and the manufacturer's recommendation. The vapor retarder should comply with the ASTM E 1745 - Class A criteria (i.e., Stego Wrap or approved equivalent), and be installed in accordance with ACI 302.1R-04 and ASTM E 1643.

- The 15-mil vapor retarder (ASTM E 1745 - Class A) shall be installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.).

- Concrete slabs, including the garage slab, should be underlain by 2 inches of clean, washed sand (SE $\geq$ 30) above a 15-mil vapor retarder (ASTM E-1745 - Class A, per Engineering Bulletin 119 [Kanare, 2005]) installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.). The manufacturer shall provide instructions for lap sealing, including minimum width of lap, method of sealing, and either supply or specify suitable products for lap sealing (ASTM E 1745), and per code.

  ACI 302.1R-04 (2004) states "If a cushion or sand layer is desired between the vapor retarder and the slab, care must be taken to protect the sand layer from taking on additional water from a source such as rain, curing, cutting, or cleaning. Wet cushion or sand layer has been directly linked in the past to significant lengthening of time required for a slab to reach an acceptable level of dryness for floor covering applications." Therefore, additional observation and/or testing will be necessary for the cushion or sand layer for moisture content, and relatively uniform thicknesses, prior to the placement of concrete.

- For very low expansive soil conditions (E.I. < 21), the vapor retarder should be underlain by 2 inches of sand (SE $\geq$ 30) placed directly on the prepared, moisture conditioned, subgrade and should be sealed to provide a continuous retarder under the entire slab, as discussed above. For higher expansive soil conditions (E.I. > 20), the vapor retarder shall be underlain by a capillary break consisting of at least 4 inches of clean crushed gravel with a maximum dimension of ¾ inch (less than 5 percent passing the No. 200 sieve).

- The maximum water to cement ratio of concrete used in foundation and slab-on-grade construction should not exceed 0.50. Additional concrete mix design recommendations should be provided by the structural consultant and/or waterproofing specialist. Concrete finishing and workablity should be addressed by the structural consultant and a waterproofing specialist.

- Where slab water/cement ratios are as indicated herein, and/or admixtures used, the structural consultant should also make changes to the concrete in the grade beams and footings in kind, so that the concrete used in the foundation and slabs are designed and/or treated for more uniform moisture protection.

- The owner should be specifically advised which areas are suitable for tile flooring, vinyl flooring, or other types of water/vapor-sensitive flooring and which areas are not suitable for these types of flooring applications. In all planned floor areas, flooring shall be installed per the manufactures recommendations.

- Additional recommendations regarding water or vapor transmission should be provided by the architect/structural engineer/slab or foundation designer and should be consistent with the specified floor coverings indicated by the architect.

Regardless of the mitigation, some limited moisture/moisture vapor transmission through the slab cannot be entirely precluded and should be anticipated. Construction crews may require special training for installation of certain product(s), as well as concrete finishing techniques. The use of specialized product(s) should be approved by the slab designer and water-proofing consultant. A technical representative of the flooring contractor should review the slab and moisture retarder plans and provide comment prior to the construction of the foundation or improvement. The vapor retarder contractor should have representatives onsite during the initial installation.

CONCRETE MASONRY UNIT (CMU) WALL DESIGN PARAMETERS (IF WARRANTED)

General

BEI (2015) includes the construction of retaining walls with a maximum exposed height of approximately 6 feet. As such, GSI is providing the following recommendations for CMU retaining wall construction. Recommendations for specialty walls (i.e., crib, earthstone, geogrid, etc.) are also included herein in a subsequent section. The use of geogrid-reinforcement in the planned retaining walls near the northerly, westerly, and easterly property lines may not be feasible without permission to grade offsite.

Conventional Retaining Walls

The design parameters provided below assume that either very low expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite materials with an E.I. up to 20 and a P.I. less than 15 are used to backfill any retaining wall.

Based on the available data, most of the onsite soils will not meet this criteria. Thus, the potential use of native backfill materials would require significant compliance testing. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the plans. Although not anticipated, any building walls, below grade (i.e., basement walls), should be water-proofed. Waterproofing should also be provided for site retaining walls in order to reduce the potential for efflorescence staining.

Preliminary Retaining Wall Foundation Design

Preliminary foundation design for retaining walls should incorporate the following recommendations:

Minimum Footing Embedment - 24 inches below the lowest adjacent grade into engineered fill overlying dense old paralic deposits. This excludes the landscape layer (upper 6 inches) and the bottom of storm water detention basins, grass swales or other areas where infiltration is planned. Retaining wall embedment should start at least 2 feet below basins or swales where infiltration is planned.

Minimum Footing Width - 24 inches

Allowable Bearing Pressure - An allowable bearing pressure of 2,500 psf may be used in the preliminary design of retaining wall foundations provided that the footing maintains a minimum width of 24 inches and extends at least 18 inches into approved engineered fill overlying dense old paralic deposits. This pressure may be increased by one-third for short-term wind and/or seismic loads.

Passive Earth Pressure - Lateral pressures in CMU retaining wall foundation design should conform to those previously provided in the "Post-Tensioned Foundation Systems" section of this report.

Lateral Sliding Resistance - A 0.35 coefficient of friction may be utilized for a concrete to soil contact when multiplied by the dead load. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

Backfill Soil Density - Soil densities ranging between 110 pcf and 115 pcf may be used in the design of retaining wall foundations. This assumes an average engineered fill compaction of at least 90 percent of the laboratory standard.

Any retaining wall footings near the perimeter of the site will likely need to be deepened into unweathered old paralic deposits for adequate vertical and lateral bearing support. All retaining wall footing setbacks from slopes should comply with the recommendations provided in the "Preliminary Foundation Design" section of this report.

Restrained Walls

Any retaining walls that will be restrained prior to placing and compacting backfill material or that have re-entrant or male corners, should be designed for an at-rest equivalent fluid pressure (EFP) of 55 pcf and 65 pcf for select and very low expansive native backfill, respectively. The design should include any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Design parameters for walls less than 3 feet in height may be superceded by San Diego Regional Standard design. 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 due to traffic, structures, seismic events or adverse geologic conditions. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request.

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)[2]	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)[3]
Level[1]	38	50
2 to 1	55	65

[1] Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall.
[2] SE ≥ 30, P.I. < 15, E.I. < 21, and ≤ 10% passing No. 200 sieve.
[3] E.I. = 0 to 20, SE ≥ 25, P.I. < 15, and ≤ 15% passing No. 200 sieve.

For preliminary planning purposes, the structural consultant should incorporate the surcharge of traffic on the back of retaining walls where vehicular traffic will occur within a horizontal distance equal to "H" from the back of any retaining wall (where "H" equals the height of the retaining wall). The traffic surcharge for light passenger cars, trucks, and vans may be taken as 100 psf/ft in the upper 5 feet of the backfill. For heavy emergency vehicle or multi-axle (HS20) truck traffic, the traffic surcharge should be 300 psf/ft in the upper 5 feet of the backfill. This does not include the surcharge of parked vehicles which should be evaluated at a higher surcharge to account for the effects of seismic loading.

Seismic Surcharge

For engineered retaining walls 6 feet or greater in overall height, retaining walls that are incorporated into a building, and/or retaining walls that may pose ingress or egress constraints to the structures, GSI recommends that the walls be evaluated for a seismic surcharge (in general accordance with 2013 CBC requirements). The site walls in this category should maintain an overturning Factor-of-Safety (FOS) of approximately 1.25 when the seismic surcharge (increment), is applied. For restrained walls, the seismic surcharge should be applied as a uniform surcharge load from the bottom of the footing (excluding shear keys) to the top of the backfill at the heel of the wall footing. This seismic surcharge pressure (seismic increment) may be taken as 15H where "H" for retained walls is the dimension previously noted as the height of the backfill to the bottom of the footing. The resultant force should be applied at a distance 0.6 H up from the bottom of the footing. For the evaluation of the seismic surcharge, the bearing pressure may exceed the static value by one-third, considering the transient nature of this surcharge. For cantilevered walls the pressure should be an inverted triangular distribution using 15H. Please note that the evaluation of the seismic surcharge is for local wall stability only.

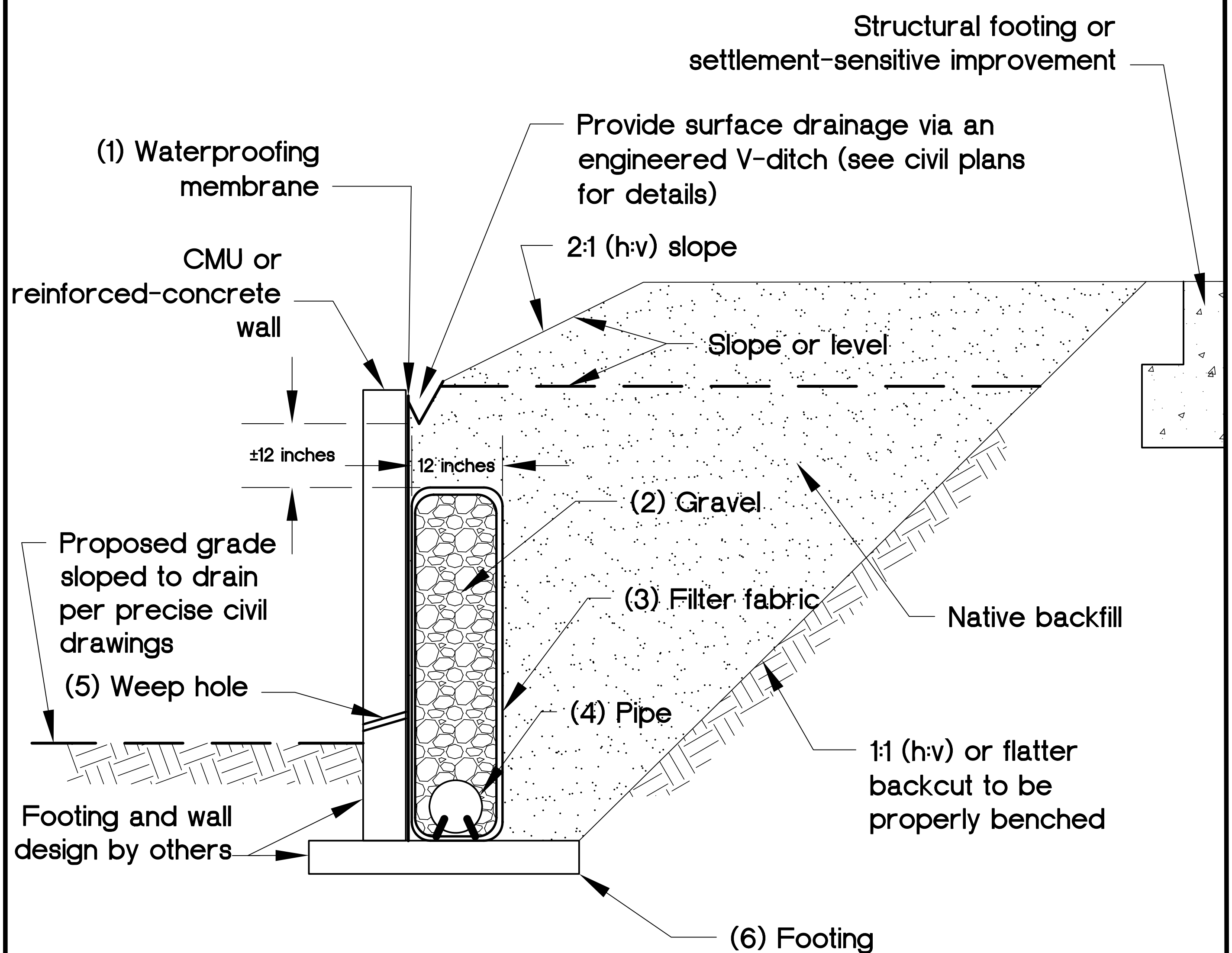
The 15H is derived from a Mononobe-Okabe solution for both restrained cantilever walls. This accounts for the increased lateral pressure due to shakedown or movement of the sand fill soil in the zone of influence from the wall or roughly a 45˚ - ϕ/2 plane away from the back of the wall. The 15H seismic surcharge is derived from the formula:

$P_h = \frac{3}{8} \bullet a_h \bullet \gamma_t H$

Where:	P_h	=	Seismic increment
	a_h	=	Probabilistic horizontal site acceleration with a percentage of "g"
	γ_t	=	total unit weight (110 to 115 pcf for site soils @ 90% relative compaction).
	H	=	Height of the wall from the bottom of the footing or point of pile fixity.

Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the backdrainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140 N or equivalent). The backdrain should flow via gravity (minimum 1 percent grade) toward an approved drainage facility. For select backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. For native backfill that has up to E.I. = 20,



(1) Waterproofing membrane.

(2) Gravel: Clean, crushed, ¾ to 1½ inch.

(3) Filter fabric: Mirafi 140N or approved equivalent.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient sloped to suitable, approved outlet point (perforations down).

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Footing: If bench is created behind the footing greater than the footing width, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

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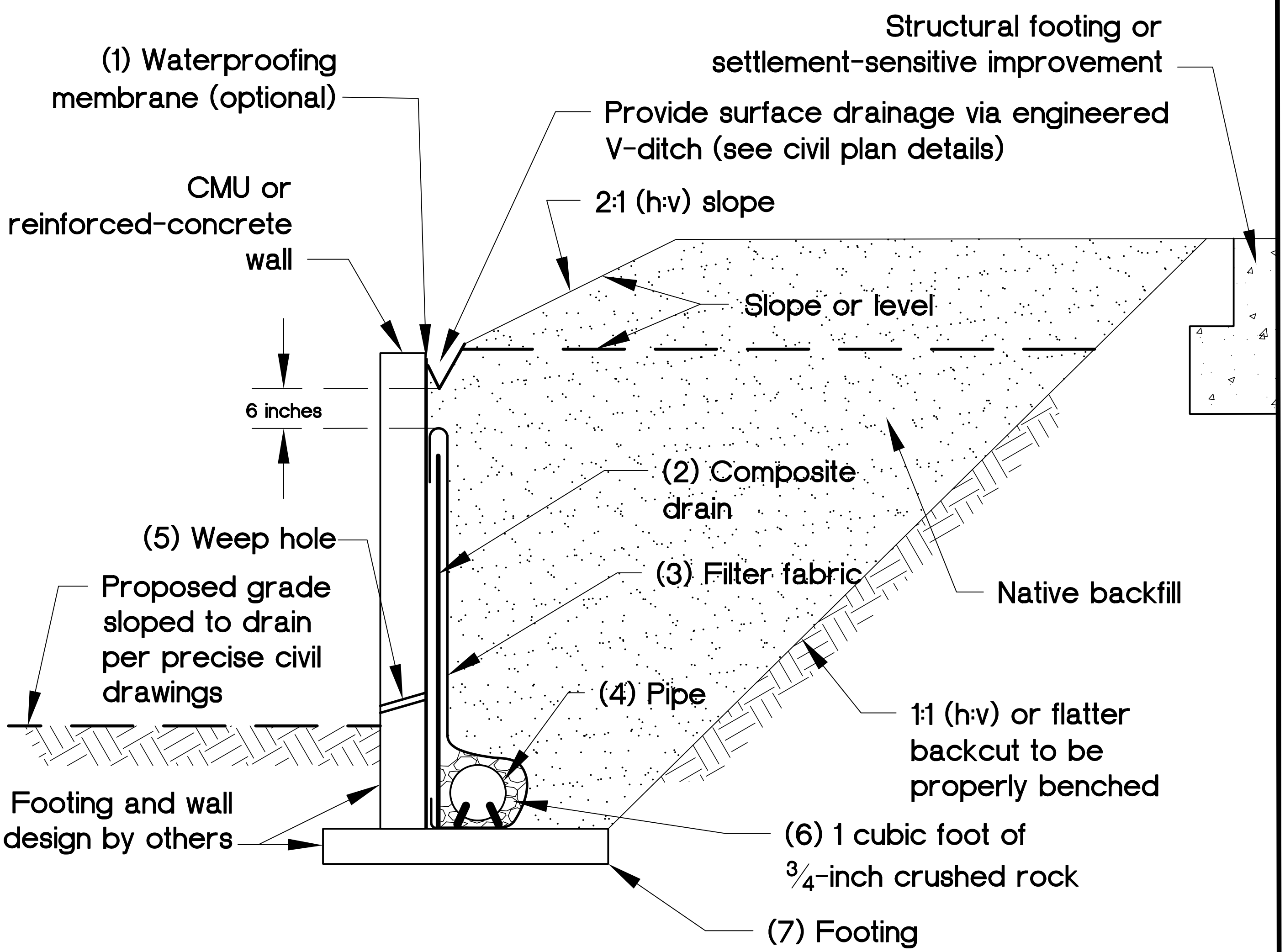
RETAINING WALL DETAIL — ALTERNATIVE A

Detail 1



(1) Waterproofing membrane (optional): Liquid boot or approved mastic equivalent.

(2) Drain: Miradrain 6000 or J-drain 200 or equivalent for non-waterproofed walls; Miradrain 6200 or J-drain 200 or equivalent for waterproofed walls (all perforations down).

(3) Filter fabric: Mirafi 140N or approved equivalent; place fabric flap behind core.

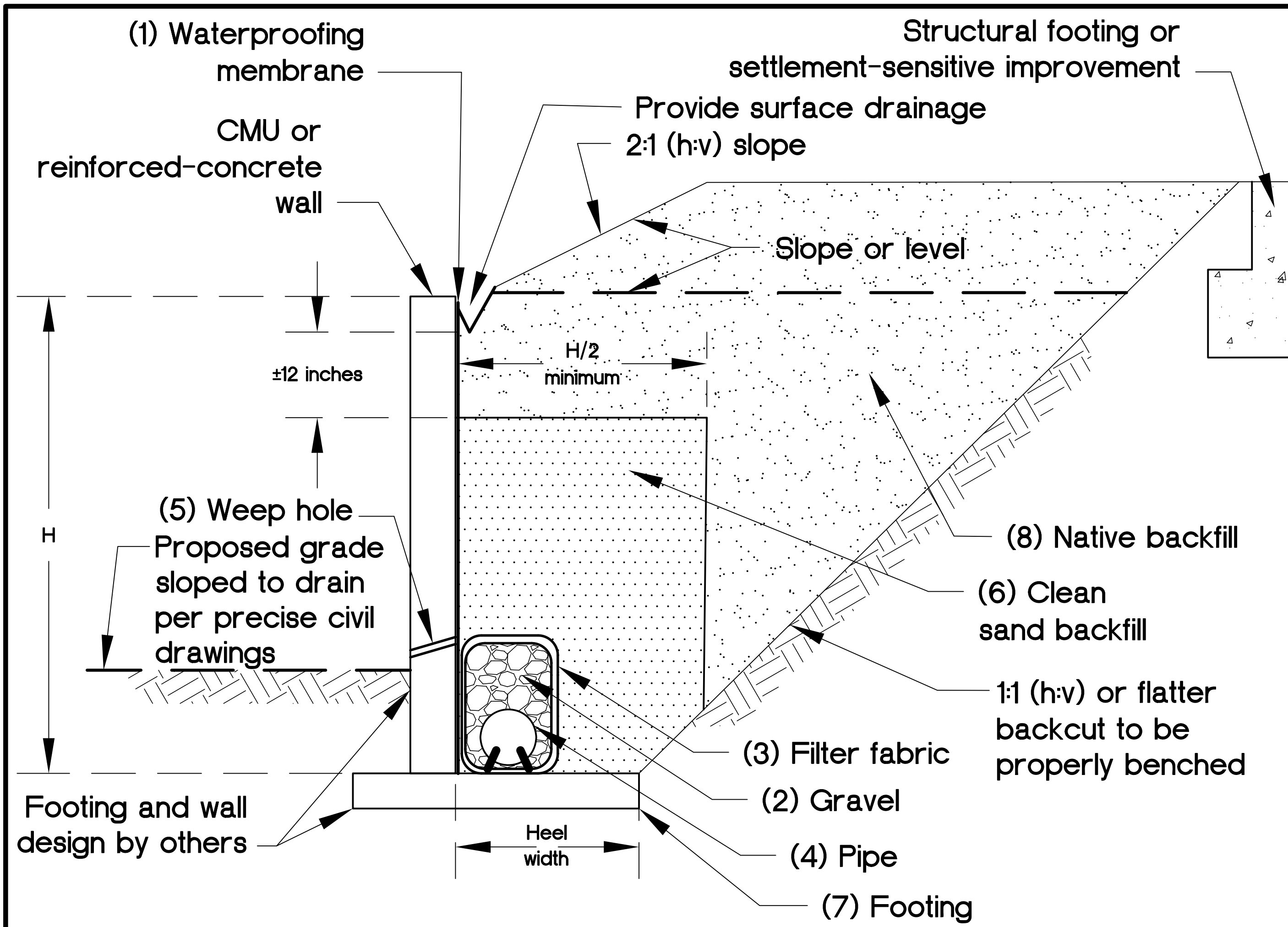
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Gravel: Clean, crushed, $^3/_4$ to $1\frac{1}{2}$ inch.

(7) Footing: If bench is created behind the footing greater than the footing width, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL — ALTERNATIVE B	Detail 2



(1) Waterproofing membrane: Liquid boot or approved masticequivalent.

(2) Gravel: Clean, crushed, ¾ to 1½ inch.

(3) Filter fabric: Mirafi 140N or approved equivalent.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Clean sand backfill: Must have sand equivalent value (S.E.) of 35 or greater; can be densified by water jetting upon approval by geotechnical engineer.

(7) Footing: If bench is created behind the footing greater than the footing width, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

(8) Native backfill: If E.I. <21 and S.E. ≥35 then all sand requirements also may not be required and will be reviewed by the geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL — ALTERNATIVE C	Detail 3

continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). Materials with an expansion index (E.I.) potential of greater than 20 should not be used as backfill for retaining walls. For more onerous expansive situations, backfill and drainage behind the retaining wall should conform with Detail 3 (Retaining Wall And Subdrain Detail Clean Sand Backfill). Retaining wall backfill should be moisture conditioned to at least the soil's optimum moisture content, placed in relatively thin lifts, and compacted to at least 90 percent of the laboratory standard (ASTM D 1557).

Outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than ±100 feet apart, with a minimum of two outlets, one on each end. The use of weep holes, only, in walls higher than 2 feet, is not recommended. The location of wall drain outlets should be shown on the grading plans. The surface of the backfill should be sealed by pavement or the top 18 inches compacted with native soil (E.I. ≤ 50). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints. Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Should wall footings transition from cut to fill, the civil designer may specify either:

a) A minimum of a 2-foot overexcavation and recompaction of cut materials for a distance of 2H, from the point of transition.

b) Increase of the amount of reinforcing steel and wall detailing (i.e., expansion joints or crack control joints) such that a angular distortion of 1/360 for a distance of 2H on either side of the transition may be accommodated. Expansion joints should be placed no greater than 20 feet on-center, in accordance with the structural engineer's/wall designer's recommendations, regardless of whether or not transition conditions exist. Expansion joints should be sealed with a flexible, non-shrink grout.

c) Embed the footings entirely into suitable old paralic deposits (i.e., deepened footings).

If transitions from cut to fill transect the wall footing alignment at an angle of less than 45 degrees (plan view), then the designer should follow recommendation "a" (above) and until such transition is between 45 and 90 degrees to the wall alignment.

TEMPORARY SHORING DESIGN AND CONSTRUCTION

Shoring of Excavations

It is our understanding that the planned development will include the construction of retaining walls near the northerly, westerly, and easterly property lines. GSI anticipates that a system of cast-in-place soldier beams and wood lagging, could be necessary to retain excavation walls if the temporary slopes, recommended herein, for retaining wall backcuts would extend into offsite property or would pass below a 1:1 projection down from the bottom outboard edge of any onsite or offsite improvements that is to remain is serviceable use during and following construction. The incorporation of tiebacks or soil nails into the shoring system may not be feasible on this site due to the close proximity of the property lines and the City of Oceanside right-of-way. If necessary, the use of internal braces and/or rakers may be used to achieve the maximum shoring height needed to complete the excavation and foundation installation.

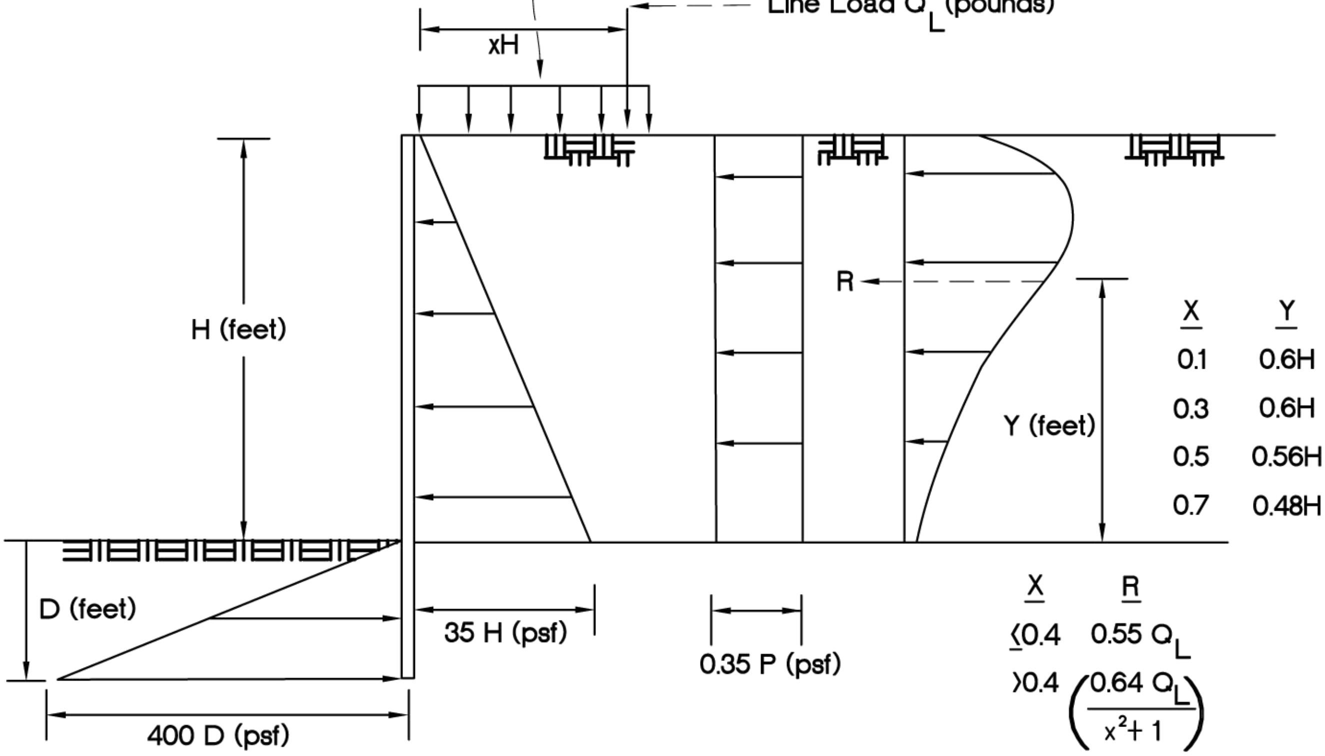
Shoring of excavations of this size is typically performed by specialty contractors with knowledge of the City of Oceanside ordinances, and current building codes, as well as the local area soil conditions. Since the design of retaining systems is sensitive to surcharge pressures behind the excavation, we recommend that this office be consulted if unusual load conditions are uncovered in the placement/installation. To that end, GSI should perform field reviews during shoring construction. Care should be exercised when excavating into the on-site soils since caving or sloughing of the earth materials is possible. Observation of soldier pile excavations and special inspections/testing should be performed during shoring construction.

Shoring of the excavation is the responsibility of the shoring contractor. Extreme caution should be used to reduce damage to any adjacent improvements caused by settlement or reduction of lateral support. Accordingly, we recommend a system of surveying and monitoring until the permanent building walls are backfilled to the design grade in order to evaluate the effects of shoring on existing onsite and offsite improvements. Pre-construction photo-documentation is also advisable. Unless incorporated into the shoring design, construction equipment storage or traffic, and/or stockpiled soils/building materials should not be stored or operated within ‘H’ feet of the top of any shored excavations (where ‘H’ equals the height of the retained earth). Temporary/permanent provisions should be made to direct any potential runoff away from the top of shored excavations. All applicable surcharges from vehicular traffic and existing structures within ‘H’ of a shored excavation should be evaluated.

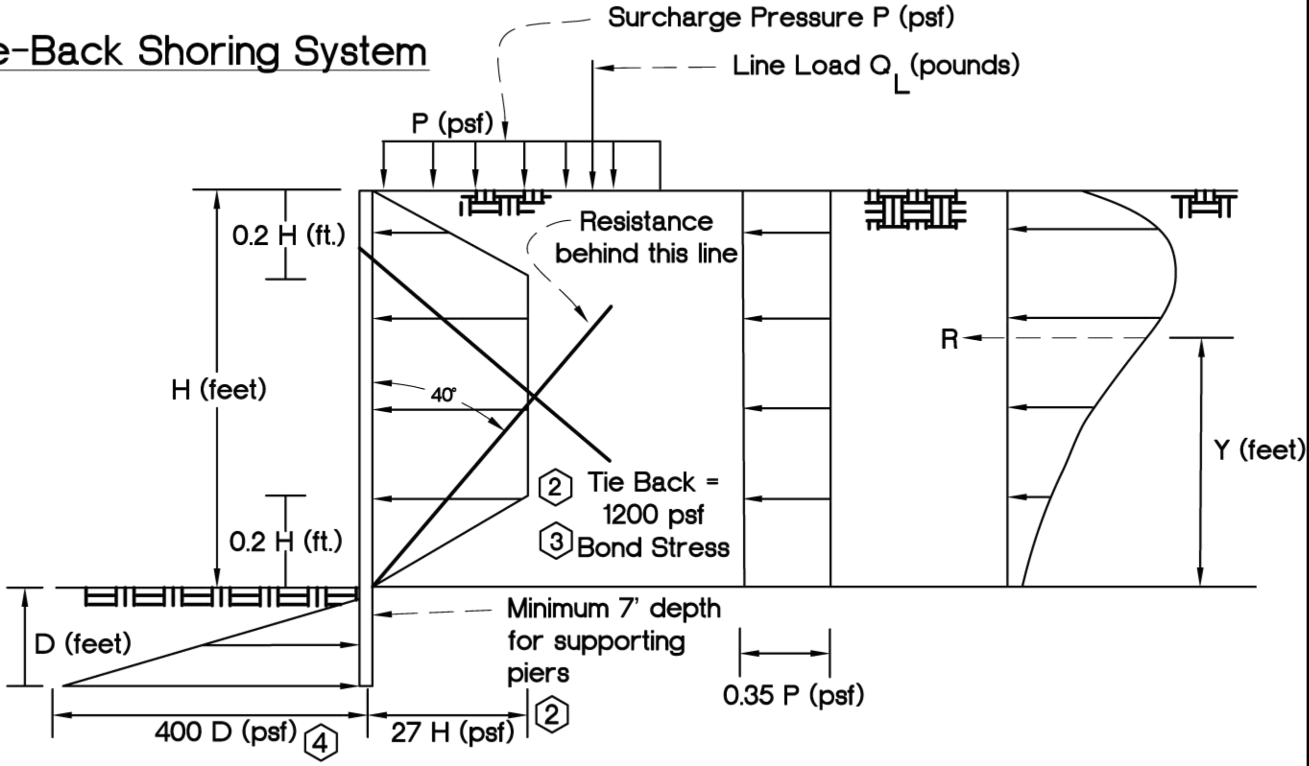
Lateral Pressure - Temporary Shoring

1. The active pressure to be utilized for the design of temporary shoring retaining level backfill conditions may be computed by the triangular pressure distribution shown in Figure 1.

Cantilever Shoring System



Tie-Back Shoring System



NOTES

1. Include groundwater effects below groundwater level.
2. Include water effects below groundwater level.
3. Grouted length greater than 7 feet; field test anchor strength.
4. Neglect passive pressure below base of excavation to a depth of one pier diameter.

GeoSoils, Inc.

RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

LATERAL EARTH PRESSURES FOR TEMPORARY SHORING SYSTEMS

Figure 1

W.O. 6912-A-SC	DATE 06/15	SCALE None

2. Passive pressure may be computed as an equivalent fluid having a given density shown in Figure 1.

3. The above criteria assumes that hydrostatic pressure is not allowed to build up behind excavation walls.

4. *Traffic Surcharge:* These recommendations are for exposed excavation walls up to 12 feet high. An empirical equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are provided for specific slope gradients of the retained material; these do not include other superimposed loading conditions such as traffic, structures, seismic events, expansive soils or adverse geologic conditions. Traffic surcharges (if applicable) for shoring walls should be minimally applied as 300 psf per lineal foot in the upper 5 feet of the shoring wall(s) if traffic is within 'H' of the back of the wall. Alternatively, for temporary shoring, this may be completed using a surface surcharge of 100 psf within 'H' of the top of the wall. It is not recommended to allow sloping surcharge (other than level backfill) within H behind the shored walls from either stockpiled soils or temporary/permanent graded slopes. Steeper slope gradients (more than level) will increase the EFP for shoring design significantly as well as associated costs. Regrading is recommended prior to shoring installation, as needed.

5. *Deflection:* The shoring system should be designed such that the maximum lateral deformation at the top of the soldier pile not exceed 1 inch. The maximum lateral deformation for the drilled pier concrete shafts at the lowest grade level should not exceed ½ inch. The point of fixity, given a CIDH diameter of 18 to 24 inches and the allowable deflection, should be on the order of 1 pile diameter from the depth of excavation (dredge line) into unweathered old paralic deposits. Lateral deflection may result in settlement of approximately ½ percent the total shoring height behind the wall.

6. Should a braced temporary shoring system be necessary, a maximum allowable bearing of 2,000 psf may be used for a temporary concrete raker footing (deadman) or by permanent lateral footings that are at least 12 inches wide by 12 inches below the lowest adjacent grade (deep) into unweathered old paralic deposits. These footings should be poured with the bearing surface normal to rakers inclined at 45 degrees. Alternatively, if a pile-supported raker is used, a passive pressure of 300 pcf may be used in the design of an 18-inch diameter cast-in-drilled hole (CIDH) pile embedded into unweathered old paralic deposits. This value may be increased by 20 percent for each additional foot of depth to a maximum lateral bearing of 3,000 psf. The coefficient of friction between concrete and unweathered old paralic deposits should be 0.35 when combined with the dead load forces.

Temporary Shoring Construction Recommendations

1. The excavation and installation of the soldier piles should be observed and documented by the project geotechnical engineer to further evaluate the geologic conditions within the influence of the temporary shoring wall and to ensure the soldier pile construction conforms to the requirements of the shoring plan.

2. Drilled excavations for soldier piles should be straight and plumb. If boulders and cobbles are encountered during drilling, the contractor should periodically recheck the drilled shaft for plumbness.

3. Although not anticipated, casing should be provided in drilled shafts if perched water and/or caving conditions are encountered during drilling operations. The bottom of the casing should be at least 4 feet below the top of the concrete as the concrete is poured and the casing is withdrawn. Although not anticipated, dewatering may be required for concrete placement if significant seepage or groundwater is encountered during construction. This should be considered during project planning.

4. The exact tip elevation of the soldier piles should be clearly indicated on the shoring plans.

5. All concrete should delivered through a tremie pipe immediately subsequent to approved excavation and steel placement. Care should be taken to prevent striking the walls of the excavations with the tremie pipe during concrete placement. Concrete should not be allowed to free fall more than 5 feet. <u>"Tailgating" concrete will not be permitted.</u>

6. Proper spacing (<u>minimum of 3 inches</u>) between H beams and the side walls, and bottoms of the drilled shafts should be provided

7. Concrete used in the shoring construction should be tested by a qualified materials testing consultant for strength and mix design.

8. Excavation for lagging should not commence until the soldier pile concrete reaches its 28-day compressive strength.

9. A complete and accurate record of all soldier pile locations, depths, concrete, strengths, quantity of concrete per pile should be maintained by the special inspector and geotechnical consultant. The shoring design engineer should be notified of any unusual conditions encountered during installation.

Monitoring of Shoring

1. The shoring designer or his designee should make periodic inspections of the job site for the purpose of observing the installation of the shoring system and monitoring of the survey.

2. Monitoring points should be established at the top of selected soldier piles and at intermediate intervals as considered appropriate by the Geotechnical Engineer.

3. Control points should be established outside the area of influence of the shoring system to ensure the accuracy of the monitoring readings.

4. Initial monitoring and photo-documentation should be performed prior to any excavation.

5. Once the excavation has commenced, periodic readings should be taken weekly until the permanent retaining wall is backfilled to the design grade. If the performance of the shoring system is within established guidelines, the shoring engineer may permit the periodic readings to be bi-weekly. Permission to conduct bi-weekly readings should be provided by the shoring design engineer in writing, and be distributed to the Geotechnical Engineer-of-Record, Structural Engineer-of-Record, Civil Engineer-of-Record, and shoring contractor. Once initiated, bi-weekly readings should continue until the permanent retaining wall is backfilled to the design grade. Thereafter, readings can be made monthly. Additional readings should be taken when requested by the special inspector, Shoring Design, Engineer, Structural Engineer-of-Record, Geotechnical Engineer-of-Record, or the Building Official.

6. Monitoring reading should be submitted to the Shoring Design Engineer, Engineer in Responsible Charge, and the Building Official (if applicable) within three business days after they are conducted. Monitoring readings should be accurate to within 0.01 feet. Results are to be submitted in tabular form showing at least the initial date of monitoring and reading, current monitoring date and reading and difference between the two readings.

7. If the total cumulative horizontal or vertical movement (from start of shoring construction) of any nearby existing improvement reaches ½-inch or soldier piles reaches 1 inch, all excavation activities should be suspended until the Geotechnical Engineer and Shoring Design Engineer determine the cause of movement. Supplemental shoring should be devised to eliminate further movement. Supplemental shoring design will require review and approval by the Building Official. Excavation should not re-commence until written permission is provided by the Building Official.

Monitoring of Existing Improvements

1. The contractor should complete written and photographic logs of any existing improvement located within 100 feet or three times the depth of shoring (whichever is greater), prior to shoring construction. A licensed surveyor should document all existing substantial cracks (i.e., greater than ⅛ inch horizontal or vertical separation) in the existing structures/improvements.

2. The contractor should document the condition of the existing improvements adjacent to the shoring wall prior to the start of shoring construction.

3. The contractor should monitor existing improvements for movement or cracking that may result from the adjacent shoring.

4. If excessive movement or visible cracking occurs, the shoring contractor should stop work and shore/reinforce the excavation, and contact the Shoring Design Engineer and the Building Official.

5. Monitoring of the existing improvements should be made at reasonable intervals as required by the registered design professional, subject to approval by the Building Official. Monitoring should be performed by a licensed surveyor.

6. Prior to commencing shoring construction, a pre-construction meeting should take place between the contractor, Shoring Design Engineer, Surveyor, Geotechnical Engineer, and the Building Official to identify monitoring locations on existing improvements.

7. If in the opinion of the Building Official or Shoring Design Engineer, monitoring data indicate excessive movement or other distress, all excavation should cease until the Geotechnical Engineer and Shoring Design Engineer investigates the situation and makes recommendations for remedial actions or continuation.

8. All readings and measurements should be submitted to the Building Official (if applicable) and Shoring Design Engineer.

TOP-OF-SLOPE WALLS/FENCES/IMPROVEMENTS

Slope Creep

Soils at the site may be expansive and therefore, may become desiccated when allowed to dry. Such soils are susceptible to surficial slope creep, especially with seasonal changes in moisture content. Typically in southern California, during the hot and dry summer period, these soils become desiccated and shrink, thereby developing surface cracks. The extent and depth of these shrinkage cracks depend on many factors such as the nature and expansivity of the soils, temperature and humidity, and extraction of moisture from surface soils by plants and roots. When seasonal rains occur, water percolates into the cracks and fissures, causing slope surfaces to expand, with a corresponding loss in soil density and shear strength near the slope surface. With the passage of time and several moisture cycles, the outer 3 to 5 feet of slope materials experience a very slow, but progressive, outward and downward movement, known as slope creep. For slope heights greater than 10 feet, this creep related soil movement will typically impact all rear yard flatwork and other secondary improvements that are located within about 15 feet from the top of slopes, such as swimming pools, concrete flatwork, etc., and in particular top of slope fences/walls. This influence is normally in the form of detrimental settlement, and tilting of the proposed improvements. The dessication/swelling and creep discussed above continues over the life of the improvements, and generally becomes progressively worse. Accordingly, the developer should provide this information to all interested/affected parties.

Top of Slope Walls/Fences

Due to the potential for slope creep for slopes higher than about 10 feet, some settlement and tilting of the walls/fence with the corresponding distresses, should be expected. To mitigate the tilting of top of slope walls/fences, we recommend that the walls/fences be constructed on deepened foundations without any consideration for creep forces, where the expansion index of the materials comprising the outer 15 feet of the slope is less than 50, or a combination of grade beam and caisson foundations, for expansion indices greater than 50 comprising the slope, with creep forces taken into account. The grade beam should be at a minimum of 12 inches by 12 inches in cross section, supported by drilled caissons, 12 inches minimum in diameter, placed at a maximum spacing of 6 feet on center, and with a minimum embedment length of 7 feet below the bottom of the grade beam. The strength of the concrete and grout should be evaluated by the structural engineer of record. The proper ASTM tests for the concrete and mortar should be provided along with the slump quantities. The concrete used should be appropriate to mitigate sulfate corrosion, as warranted. The design of the grade beam and caissons should be in accordance with the recommendations of the project structural engineer, and include the utilization of the following geotechnical parameters:

Creep Zone: 5-foot vertical zone below the slope face and projected upward parallel to the slope face.

Creep Load: The creep load projected on the area of the grade beam should be taken as an equivalent fluid approach, having a density of 60 pcf. For the caisson, it should be taken as a uniform 900 pounds per linear foot of caisson's depth, located above the creep zone.

Point of Fixity: Located a distance of 1.5 times the caisson's diameter, below the creep zone.

Passive Resistance: Passive earth pressure of 300 psf per foot of depth per foot of caisson diameter, to a maximum value of 4,500 psf may be used to determine caisson depth and spacing, provided that they meet or exceed the minimum requirements stated above. To determine the total lateral resistance, the contribution of the creep prone zone above the point of fixity, to passive resistance, should be disregarded.

Allowable Axial Capacity:

Shaft capacity : 300 psf applied below the point of fixity over the surface area of the shaft.

Tip capacity: 3,000 psf for a caisson bearing on approved engineered fill or unweathered old paralic deposits. This assumes that the bottom of the drilled shaft is cleaned of loose materials and debris.

EXPANSIVE SOILS, DRIVEWAY, FLATWORK, AND OTHER IMPROVEMENTS

The soil materials on site are expansive. The effects of expansive soils are cumulative, and typically occur over the lifetime of any improvements. On relatively level areas, when the soils are allowed to dry, the dessication and swelling process tends to cause heaving and distress to flatwork and other improvements. The resulting potential for distress to improvements may be reduced, but not totally eliminated. To that end, it is recommended that the developer should notify all interested/affected parties of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. The subgrade area for concrete slabs should be compacted to achieve a minimum 90 percent relative compaction, and then be presoaked to 2 to 3 percentage points above (or 125 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.

2. Concrete slabs should be cast over a relatively non-yielding surface, consisting of a 4-inch layer of crushed rock, gravel, or clean sand, that should be compacted and level prior to pouring concrete. The layer should wet-down completely prior to pouring concrete, to minimize loss of concrete moisture to the surrounding earth materials.

3. Exterior slabs should be a minimum of 4 inches thick. Driveway slabs and approaches should additionally have a thickened edge (12 inches) adjacent to all landscape areas, to help impede infiltration of landscape water under the slab.

4. The use of transverse and longitudinal control joints are recommended to help control slab cracking due to concrete shrinkage or expansion. Two ways to mitigate such cracking are: a) add a sufficient amount of reinforcing steel, increasing tensile strength of the slab; and, b) provide an adequate amount of control and/or expansion joints to accommodate anticipated concrete shrinkage and expansion.

   In order to reduce the potential for unsightly cracks, slabs should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. The exterior slabs should be scored or saw cut, ½ to ⅜ inches deep, often enough so that no section is greater than 10 feet by 10 feet. For sidewalks or narrow slabs, control joints should be provided at intervals of every 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

5. No traffic should be allowed upon the newly poured concrete slabs until they have been properly cured to within 75 percent of design strength. Concrete compression strength should be a minimum of 2,500 psi.

6. Driveways, sidewalks, and patio slabs adjacent to the buildings should be separated from the structure with thick expansion joint filler material. In areas directly adjacent to a continuous source of moisture (i.e., irrigation, planters, etc.), all joints should be additionally sealed with flexible mastic.

7. Planters and walls should not be tied to the buildings.

8. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied in at least two directions.

9. Any masonry landscape walls that are to be constructed throughout the property should be grouted and articulated in segments no more than 20 feet long. These segments should be keyed or doweled together.

10. Utilities should be enclosed within a closed utilidor (vault) or designed with flexible connections to accommodate differential settlement and expansive soil conditions.

11. Positive site drainage should be maintained at all times. Finish grade on the lots should provide a minimum of 1 to 2 percent fall to the street, as indicated herein. It should be kept in mind that drainage reversals could occur, including post-construction settlement, if relatively flat yard drainage gradients are not periodically maintained by the owner.

12. Due to expansive soils, air conditioning (A/C) units should be supported by slabs that are incorporated into the building foundation or constructed on a rigid slab with flexible couplings for plumbing and electrical lines. A/C waste water lines should be drained to a suitable non-erosive outlet.

13. Shrinkage cracks could become excessive if proper finishing and curing practices are not followed. Finishing and curing practices should be performed per the Portland Cement Association Guidelines. Mix design should incorporate rate of curing for climate and time of year, sulfate content of soils, corrosion potential of soils, and fertilizers used on site.

PRELIMINARY ASPHALTIC CONCRETE OVER AGGREGATE BASE SECTIONS - DIXIE AND GRACE STREETS

The recommended asphaltic concrete over aggregate base sections (AC/AB) within the Dixie and Grace Street right-of-ways previously provided in GSI (2007) are repeated below.

TRAFFIC AREA	TRAFFIC INDEX	SUBGRADE R-VALUE	AC THICKNESS (INCHES)	AGGREGATE BASE THICKNESS(2) (INCHES)
Dixie and Grace Streets	5.0	9	3.0(1)	9.0
Dixie Street (Stations 19^{+52} through 20^{+00} [if feasible])	5.0	13	3.0(1)	8.5
(1) City of Oceanside minimum thickness (2) Denotes Class 2 Aggregate Base ($R \geq 78$, SE ≥ 22)				

The thickness of the aggregate base section may be reduced by thickening the asphaltic concrete section; creating a more balanced pavement section, as indicated in the following table:

TRAFFIC AREA	TRAFFIC INDEX	SUBGRADE R-VALUE	AC THICKNESS (INCHES)	AGGREGATE BASE THICKNESS(1) (INCHES)
Dixie and Grace Streets	5.0	9	4.0	7.0
Dixie Street (Stations 19^{+52} through 20^{+00} [if feasible])	5.0	13	4.0	6.0(2)
(1) Denotes Class 2 Aggregate Base ($R \geq 78$, SE ≥ 22) (2) City of Oceanside minimum thickness				

All pavement installation, including preparation and compaction of subgrade, compaction of base material, and placement and rolling of asphaltic concrete, should be done in accordance with City of Oceanside guidelines and under the observation and testing services provided by the project geotechnical engineer and/or the City.

The recommended pavement sections provided above are intended as minimum guidelines. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the T.I. used for design, increased maintenance and repair could be required for the pavement section. Consideration should be given to the increased potential for distress from overuse of paved street areas by heavy equipment and/or construction related heavy traffic (e.g., concrete trucks, loaded supply trucks, etc.), particularly when the final section is not in place (i.e., topcoat). Best management construction practices should be followed at all times, especially during inclement weather.

PAVEMENT GRADING RECOMMENDATIONS

General

All section changes should be properly transitioned. If adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. A GSI representative should be present for the preparation of subgrade, base rock, and asphalt concrete.

Subgrade

Within street and parking areas, all surficial deposits of loose soil material should be removed and recompacted as recommended. After the loose soils are removed, the bottom is to be scarified to a depth of at least 6 inches, moisture conditioned as necessary and compacted to 95 percent of the maximum laboratory density (ASTM D-1557) or the City minimum, as determined by ASTM D-1557.

Deleterious material, excessively wet or dry pockets, concentrated zones of oversized rock fragments, and any other unsuitable materials encountered during grading should be removed. The compacted fill material should then be brought to the elevation of the proposed subgrade for the pavement. The subgrade should be proof-rolled in order to ensure a uniform firm and unyielding surface. All grading and fill placement should be observed by the project soil engineer and/or his representative.

Aggregate Base Rock

Compaction tests are required for the recommended base section. Minimum relative compaction required will be 95 percent of the laboratory maximum density as determined by ASTM test method D-1557 and/or Caltrans Test Method Number California 216. Base aggregate should be in accordance with the Caltrans Class 2 base rock (minimum R-value=78).

Paving

Prime coat may be omitted if all of the following conditions are met:

1. The asphalt pavement layer is placed within two weeks of completion of base and/or subbase course.

2. Traffic is not routed over completed base before paving

3. Construction is completed during the dry season of May through October.

4. The base is kept free of debris prior to placement of asphaltic concrete.

If construction is performed during the wet season of November through April, prime coat may be omitted if no rain occurs between completion of base course and paving <u>and</u> the time between completion of base and paving is reduced to three days, provided the base is free of loose soil or debris. Where prime coat has been omitted and rain occurs, traffic is routed over base course, or paving is delayed, measures shall be taken to restore base course, and subgrade to conditions that will meet specifications as directed by the soil engineer.

Drainage

Positive drainage should be provided for all surface water to drain towards the curb and gutter, or to an approved drainage channel. Positive site drainage should be maintained at all times. Water should not be allowed to pond or seep into the ground. If planters or landscaping are adjacent to paved areas, measures should be taken to minimize the potential for water to enter the pavement section, such as thickened edges, cut-offs wall, french drains, etc. GSI recommends that drainage, within the project area, should be designed to <u>not</u> saturate subgrade soils within Dixie and Grace Streets. The Client should consider the use of thickened curbs, cut-off walls, or french drains along Dixie and Grace Streets in order to reduce the potential for saturation of subgrade soils. The thickened curbs, cut-off walls, or french drains should extend at least 1 foot below the subgrade elevation. R-value testing indicates that the subgrade soils underlying these streets may deflect considerably under load when wet potentially resulting in pavement distress and reduced life span. If french drains are used, the pipe should be tightlined to a suitable outlet as approved by the City of Oceanside. GSI will provide additional recommendations for french drains if this option is requested.

ONSITE INFILTRATION-RUNOFF RETENTION SYSTEMS

General

Onsite infiltration-runoff retention systems (OIRRS) are typically required for Low Impact Development (LID) Best Management Practices (BMPs) for development projects. To that end, some guidelines should/must be followed in the planning, design, and construction of such systems. Such facilities, if improperly designed or implemented without consideration of the geotechnical aspects of site conditions, can contribute to flooding, saturation of bearing materials beneath site improvements, slope instability, and possible concentration and contribution of pollutants into the groundwater or storm drain and/or utility trench systems.

A key factor in these systems is the infiltration rate (often referred to as the percolation rate) which can be ascribed to, or determined for, the earth materials within which these systems are installed. Additionally, the infiltration rate of the designed system (which may include gravel, sand, mulch/topsoil, or other amendments, etc.) will need to be considered. The project infiltration testing is very site specific, any changes to the location of the proposed OIRRS and/or estimated size of the OIRRS, may require additional infiltration testing. GSI anticipates that relatively impermeable engineered fill and/or old paralic deposits will occur near the surface at the conclusion of grading.

Some of the methods which are utilized for onsite infiltration include percolation basins, dry wells, bio-swale/bio-retention, permeable pavers/pavement, infiltration trenches, filter boxes and subsurface infiltration galleries/chambers. Some of these systems are constructed using native and import soils, perforated piping, and filter fabrics while others employ structural components such as storm water infiltration chambers and filters/separators. Every site will have characteristics which should lend themselves to one or more of these methods; but, not every site is suitable for OIRRS. In practice, OIRRS are usually initially designed by the project design civil engineer. Selection of methods should include (but should not be limited to) review by licensed professionals including the geotechnical engineer, hydrogeologist, engineering geologist, project civil engineer, landscape architect, environmental professional, and industrial hygienist. Applicable governing agency requirements should be reviewed and included in design considerations.

The following geotechnical guidelines should be considered when designing onsite infiltration-runoff retention systems:

- Based on our review of the United States Department of Agriculture Soil Survey (http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx), the onsite soils consist of the Chesterton fine sandy loam, 2 to 5 percent slopes. The capacity of the most limiting layer to transmit water (Ksat) for this mapped soil unit is characterized as very low to moderately low (0.00 to 0.06 inches per hour [in/hr]). This mapped soil unit fall into Hydrologic Soil Group (HSG) “D.” County of

San Diego (2007) indicates that HSG "D" soils have very slow infiltration rates where infiltration may not be feasible.

- It is not good engineering practice to allow water to saturate soils, especially near slopes or improvements; however, the controlling agency/authority is now requiring this for OIRRS purposes on many projects.

- If infiltration is planned, infiltration system design should be based on actual infiltration testing results/data, preferably utilizing double-ring infiltrometer testing (ASTM D 3385) to determine the infiltration rate of the earth materials being contemplated for infiltration.

- Wherever possible, infiltration systems should not be installed within ±50 feet of the tops of slopes steeper than 15 percent or within H/3 from the tops of slopes (where H equals the height of slope).

- Wherever possible, infiltrations systems should not be placed within a distance of H/2 from the toes of slopes (where H equals the height of slope).

- Impermeable liners and subdrains should be used along the bottom of bioretention swales/basins located within the influence of slopes.

- Impermeable liners and subdrains should be used along the sides and bottom of bioretention swales/basins located within the influence of slopes. Impermeable liners should consist of a 30-mil polyvinyl chloride (PVC) membrane with the following properties:

  Specific Gravity (ASTM D792): 1.2 (g/cc, min.); Tensile (ASTM D882): 73 (lb/in-width, min); Elongation at Break (ASTM D882): 380 (%, min); Modulus (ASTM D882): 30 (lb/in-width, min.); and Tear Strength (ASTM D1004): 8 (lb/in, min); Seam Shear Strength (ASTM D882) 58.4 (lb/in, min); Seam Peel Strength (ASTM D882) 15 (lb/in, min).

  Subdrains should consist of at least 4-inch diameter Schedule 40 or SDR 35 drain pipe with perforations oriented down. The drain pipe should be sleeved with a filter sock.

- The landscape architect should be notified of the location of the proposed OIRRS. If landscaping is proposed <u>within</u> the OIRRS, consideration should be given to the type of vegetation chosen and their potential effect upon subsurface improvements (i.e., some trees/shrubs will have an effect on subsurface improvements with their extensive root systems). Over-watering landscape areas above, or adjacent to, the proposed OIRRS could adversely affect performance of the system.

- Areas adjacent to, or within, the OIRRS that are subject to inundation should be properly protected against scouring, undermining, and erosion, in accordance with the recommendations of the design engineer.

- Seismic shaking may result in the formation of a seiche which could potential overtop the banks of an OIRRS and result in down-gradient flooding and scour.

- If subsurface infiltration galleries/chambers are proposed, the appropriate size, depth interval, and ultimate placement of the detention/infiltration system should be evaluated by the design engineer, and be of sufficient width/depth to achieve optimum performance, based on the infiltration rates provided. In addition, proper debris filter systems will need to be utilized for the infiltration galleries/chambers. Debris filter systems will need to be self cleaning and periodically and regularly maintained on a regular basis. Provisions for the regular and periodic maintenance of any debris filter system is recommended and this condition should be disclosed to all interested/affected parties.

- Infiltrations systems should not be installed within ±8 feet of building foundations utility trenches, and walls, or a 1:1 (h:v) slope (down and away) from the bottom elements of these improvements. Alternatively, deepened foundations and/or pile/pier supported improvements may be used.

- Infiltrations systems should not be installed adjacent to pavement and/or hardscape improvements. Alternatively, deepened/thickened edges and curbs and/or impermeable liners may be utilized in areas adjoining the OIRRS.

- As with any OIRRS, localized ponding and groundwater seepage should be anticipated. The potential for seepage and/or perched groundwater to occur after site development should be disclosed to all interested/affected parties.

- Installation of infiltrations systems should avoid expansive soils (E.I. ≥51) or soils with a relatively high plasticity index (P.I. > 20).

- Infiltration systems should not be installed where the vertical separation of the groundwater level is less than ±10 feet from the base of the system.

- Where permeable pavements are planned as part of the system, the site Traffic Index (T.I.) should be less than 25,000 Average Daily Traffic (ADT), as recommended in Allen, et al. (2011).

- Infiltration systems should be designed using a suitable factor of safety (FOS) to account for uncertainties in the known infiltration rates (as generally required by the controlling authorities), and reduction in performance over time.

- As with any OIRRS, proper care will need to be provided. Best management practices should be followed at all times, especially during inclement weather. Provisions for the management of any siltation, debris within the OIRRS, and/or overgrown vegetation (including root systems) should be considered. An appropriate inspection schedule will need to adopted and provided to all interested/affected parties.

- Any designed system will require regular and periodic maintenance, which may include rehabilitation and/or complete replacement of the filter media (e.g., sand, gravel, filter fabrics, topsoils, mulch, etc.) or other components utilized in construction, so that the design life exceeds 15 years. Due to the potential for piping and adverse seepage conditions, a burrowing rodent control program should also be implemented onsite.

- All or portions of these systems may be considered attractive nuisances. Thus, consideration of the effects of, or potential for, vandalism should be addressed.

- Newly established vegetation/landscaping (including phreatophytes) may have root systems that will influence the performance of the OIRRS or nearby LID systems.

- The potential for surface flooding, in the case of system blockage, should be evaluated by the design engineer.

- Any proposed utility backfill materials (i.e., inlet/outlet piping and/or other subsurface utilities) located within or near the proposed area of the OIRRS may become saturated. This is due to the potential for piping, water migration, and/or seepage along the utility trench line backfill. If utility trenches cross and/or are proposed near the OIRRS, cut-off walls or other water barriers will need to be installed to mitigate the potential for piping and excess water entering the utility backfill materials. Planned or existing utilities may also be subject to piping of fines into open-graded gravel backfill layers unless separated from overlying or adjoining OIRRS by geotextiles and/or slurry backfill.

- The use of OIRRS above existing utilities that might degrade/corrode with the introduction of water/seepage should be avoided.

- A vector control program may be necessary as stagnant water contained in OIRRS may attract mammals, birds, and insects that carry pathogens.

DEVELOPMENT CRITERIA

Slope Deformation

Compacted fill slopes designed using customary factors-of-safety for gross or surficial stability and constructed in general accordance with the design specifications should be expected to undergo some differential vertical heave or settlement in combination with differential lateral movement in the out-of-slope direction, after grading. This post-construction movement occurs in two forms: slope creep, and lateral fill extension (LFE). Slope creep is caused by alternate wetting and drying of the fill soils which results in slow downslope movement. This type of movement is expected to occur throughout the life of the slope, and is anticipated to potentially affect improvements or structures (e.g., separations and/or cracking), placed near the top-of-slope, up to a maximum distance of approximately 15 feet from the top-of-slope, depending on the slope height. This movement generally results in rotation and differential settlement of improvements located within the creep zone. LFE occurs due to deep wetting from irrigation and rainfall on slopes comprised of expansive materials. Although some movement should be expected, long-term movement from this source may be minimized, but not eliminated, by placing the fill throughout the slope region, wet of the fill's optimum moisture content.

It is generally not practical to attempt to eliminate the effects of either slope creep or LFE. Suitable mitigative measures to reduce the potential of lateral deformation typically include: setback of improvements from the slope faces (per the 2013 CBC), positive structural separations (i.e., joints) between improvements, and stiffening and deepening of foundations. Expansion joints in walls should be placed no greater than 20 feet on-center, and in accordance with the structural engineer's recommendations. All of these measures are recommended for design of structures and improvements. The ramifications of the above conditions, and recommendations for mitigation, should be provided to all interested/affected parties.

Slope Maintenance and Planting

Water has been shown to weaken the inherent strength of all earth materials. Slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Over-watering should be avoided as it adversely affects site improvements, and causes perched groundwater conditions. Graded slopes constructed utilizing onsite materials would be erosive. Eroded debris may be minimized and surficial slope stability enhanced by establishing and maintaining a suitable vegetation cover soon after construction. Compaction to the face of fill slopes would tend to minimize short-term erosion until vegetation is established. Plants selected for landscaping should be light weight, deep rooted types that require little water and are capable of surviving the prevailing climate. Jute-type matting or other fibrous covers may aid in allowing the establishment of a sparse plant cover. Utilizing plants other than those recommended above will increase the potential for perched water, staining, mold, etc., to develop. A rodent control program to prevent burrowing should be implemented. Irrigation of natural (ungraded) slope areas is generally not recommended. These recommendations regarding plant type, irrigation practices, and rodent control should be provided to all interested/affected parties. Over-steepening of slopes should be avoided during building construction activities and landscaping.

Drainage

Adequate surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to mitigate ponding of water anywhere on the property, and especially near structures and tops of slopes. Surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage within the property should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and tops of slopes, and not allowed to pond and/or seep into the ground. In general, site drainage should conform to Section 1804.3 of the 2013 CBC. Consideration should be given to avoiding construction of planters adjacent to the building. Building pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, down spouts, or other appropriate means may be utilized to control roof drainage. Down spouts, or drainage devices should outlet a minimum of 5 feet from structures or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for minimizing this effect could be provided upon request.

Erosion Control

Onsite earth materials have a moderate to high erosion potential. Consideration should be given to providing hay bales and silt fences for the temporary control of surface water, from a geotechnical viewpoint.

Landscape Maintenance

Only the amount of irrigation necessary to sustain plant life should be provided. Over-watering the landscape areas will adversely affect proposed site improvements. We would recommend that any proposed open-bottom planters adjacent to the structure be eliminated for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be utilized. An outlet placed in the bottom of the planter, could be installed to direct drainage away from structures or any exterior concrete flatwork. If planters are constructed adjacent to the structure, the sides and bottom of the planter should be provided with a moisture barrier to prevent penetration of irrigation water into the subgrade. Provisions should be made to drain the excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters. Consideration should be given to the type of vegetation chosen and their potential effect upon surface improvements (i.e., some trees will have an effect on concrete flatwork with their extensive root systems). From a geotechnical standpoint leaching is not recommended for establishing landscaping. If the surface soils are processed for the purpose of adding amendments, they should be recompacted to 90 percent minimum relative compaction.

Gutters and Downspouts

As previously discussed in the drainage section, the installation of gutters and downspouts should be considered to collect roof water that may otherwise infiltrate the soils adjacent to the structures. If utilized, the downspouts should be drained into PVC collector pipes or other non-erosive devices (e.g., paved swales or ditches; below grade, solid tight-lined PVC pipes; etc.), that will carry the water away from the building, to an appropriate outlet, in accordance with the recommendations of the design civil engineer. Downspouts and gutters are not a requirement; however, from a geotechnical viewpoint, provided that positive drainage is incorporated into project design (as discussed previously).

Subsurface and Surface Water

Subsurface and surface water are not anticipated to affect site development, provided that the recommendations contained in this report are incorporated into final design and construction and that prudent surface and subsurface drainage practices are incorporated into the construction plans. Perched groundwater conditions along zones of contrasting permeabilities may not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities, and should be anticipated. Should perched groundwater conditions develop, this office could assess the affected area(s) and provide the appropriate recommendations to mitigate the observed groundwater conditions. Groundwater conditions may change with the introduction of irrigation, rainfall, or other factors.

Site Improvements

If in the future, any additional improvements (e.g., pools, spas, etc.) are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. Pools and/or spas should not be constructed without specific design and construction recommendations from GSI, and this construction recommendation should be provided to all interested/affected parties. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, utility trench and retaining wall backfills, flatwork, etc.

Tile Flooring

Tile flooring can crack, reflecting cracks in the concrete slab below the tile, although small cracks in a conventional slab may not be significant. Therefore, the designer should consider additional steel reinforcement for concrete slabs-on-grade where tile will be placed. The tile installer should consider installation methods that reduce possible cracking of the tile such as slipsheets. Slipsheets or a vinyl crack isolation membrane (approved by the Tile Council of America/Ceramic Tile Institute) are recommended between tile and concrete slabs on grade.

Additional Grading

This office should be notified in advance of any fill placement, supplemental regrading of the site, or trench backfilling after rough grading has been completed. This includes completion of grading in the street, driveway approaches, driveways, parking areas, and utility trench and retaining wall backfills.

Footing Trench Excavation

All footing excavations should be observed by a representative of this firm subsequent to trenching and prior to concrete form and reinforcement placement. The purpose of the observations is to evaluate that the excavations have been made into the recommended bearing material and to the minimum widths and depths recommended for construction. If loose or compressible materials are exposed within the footing excavation, a deeper footing or removal and recompaction of the subgrade materials would be recommended at that time. Footing trench spoil and any excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent, if not removed from the site.

Trenching/Temporary Construction Backcuts

Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees [except as specifically superceded within the text of this report]), should be anticipated. All excavations should be observed by an engineering geologist or soil engineer from GSI, prior to workers entering the excavation or trench, and minimally conform to CAL-OSHA, state, and local safety codes. Should adverse conditions exist, appropriate recommendations would be offered at that time. The above recommendations should be provided to any contractors and/or subcontractors, or owner(s), etc., that may perform such work.

Utility Trench Backfill

1. All interior utility trench backfill should be brought to at least 2 percent above optimum moisture content and then compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard. As an alternative for shallow (12-inch to 18-inch) under-slab trenches, sand having a sand equivalent value of

30 or greater may be utilized and jetted or flooded into place. Observation, probing and testing should be provided to evaluate the desired results.

2. Exterior trenches adjacent to, and within areas extending below a 1:1 plane projected from the outside bottom edge of the footing, and all trenches beneath hardscape features and in slopes, should be compacted to at least 90 percent of the laboratory standard. Sand backfill, unless excavated from the trench, should not be used in these backfill areas. Compaction testing and observations, along with probing, should be accomplished to evaluate the desired results.

3. All trench excavations should conform to CAL-OSHA, state, and local safety codes.

4. Utilities crossing grade beams, perimeter beams, or footings should either pass below the footing or grade beam utilizing a hardened collar or foam spacer, or pass through the footing or grade beam in accordance with the recommendations of the structural engineer.

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

We recommend that observation and/or testing be performed by GSI at each of the following construction stages:

- During grading.
- During excavation.
- During placement of subdrains or other subdrainage devices, prior to placing fill and/or backfill.
- After excavation of building footings, retaining wall footings, and free standing walls footings, prior to the placement of reinforcing steel or concrete.
- Prior to pouring any slabs or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor retarders (i.e., visqueen, etc.).
- During retaining wall subdrain installation, prior to backfill placement.
- During placement of backfill for area drain, interior plumbing, utility line trenches, and retaining wall backfill.

- During any slope construction/repair.

- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.

- When any owner improvements, such as flatwork, spas, pools, walls, etc., are constructed, prior to construction.

- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, and/or to comply with code requirements.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

The design civil engineer, structural engineer, post-tension designer, architect, landscape architect, wall designer, etc., should review the recommendations provided herein, incorporate those recommendations into all their respective plans, and by explicit reference, make this report part of their project plans. This report presents minimum design criteria for the design of slabs, foundations and other elements possibly applicable to the project. These criteria should not be considered as substitutes for actual designs by the structural engineer/designer. Please note that the recommendations contained herein are not intended to preclude the transmission of water or vapor through the slab or foundation. The structural engineer/foundation and/or slab designer should provide recommendations to not allow water or vapor to enter into the structure so as to cause damage to another building component, or so as to limit the installation of the type of flooring materials typically used for the particular application.

The structural engineer/designer should analyze actual soil-structure interaction and consider, as needed, bearing, expansive soil influence, and strength, stiffness and deflections in the various slab, foundation, and other elements in order to develop appropriate, design-specific details. As conditions dictate, it is possible that other influences will also have to be considered. The structural engineer/designer should consider all applicable codes and authoritative sources where needed. If analyses by the structural engineer/designer result in less critical details than are provided herein as minimums, the minimums presented herein should be adopted. It is considered likely that some, more restrictive details will be required.

If the structural engineer/designer has any questions or requires further assistance, they should not hesitate to call or otherwise transmit their requests to GSI. In order to mitigate potential distress, the foundation and/or improvement's designer should confirm to GSI and the governing agency, in writing, that the proposed foundations and/or improvements can tolerate the amount of differential settlement and/or expansion characteristics and other design criteria specified herein.

PLAN REVIEW

Final project plans (grading, precise grading, foundation, retaining wall, landscaping, etc.), should be reviewed by this office prior to construction, so that construction is in accordance with the conclusions and recommendations of this report. Based on our review, supplemental recommendations and/or further geotechnical studies may be warranted.

LIMITATIONS

The materials encountered on the project site and utilized for our analysis are believed representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during mass grading. Site conditions may vary due to seasonal changes or other factors.

Inasmuch as our study is based upon our review and engineering analyses and laboratory data, the conclusions and recommendations are professional opinions. These opinions have been derived in accordance with current standards of practice, and no warranty, either express or implied, is given. Standards of practice are subject to change with time. GSI assumes no responsibility or liability for work or testing performed by others, or their inaction; or work performed when GSI is not requested to be onsite, to evaluate if our recommendations have been properly implemented. Use of this report constitutes an agreement and consent by the user to all the limitations outlined above, notwithstanding any other agreements that may be in place. In addition, this report may be subject to review by the controlling authorities. Thus, this report brings to completion our scope of services for this portion of the project. All samples will be disposed of after 30 days, unless specifically requested by the client, in writing.

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

American Concrete Institute, Committee 318, 2011, Building code requirements for structural concrete (ACI318-11) and commentary, dated August.

American Concrete Institute (ACI) Committee 302, 2004, Guide for concrete floor and slab construction, ACI 302.1R-04, dated June.

Allen, V., Connerton, A., and Carlson, C., 2011, Introduction to Infiltration Best Management Practices (BMP), Contech Construction Products, Inc., Professional Development Series, dated December.

American Society for Testing and Materials (ASTM), 2003, Standard test method for infiltration rate of soils in field using double-ring infiltrometer, Designation D 3385-03, dated August.

_____,1998, Standard practice for installation of water vapor retarder used in contact with earth or granular fill under concrete slabs, Designation: E 1643-98 (Reapproved 2005).

_____, 1997, Standard specification for plastic water vapor retarders used in contact with soil or granular fill under concrete slabs, Designation: E 1745-97 (Reapproved 2004).

American Society of Civil Engineers, 2006, Minimum design loads for buildings and other structures, ASCE Standard ASCE/SEI 7-05.

Blake, Thomas F., 2000a, EQFAULT, A computer program for the estimation of peak horizontal acceleration from 3-D fault sources; Windows 95/98 version.

_____, 2000b, EQSEARCH, A computer program for the estimation of peak horizontal acceleration from California historical earthquake catalogs; Updated to July 2013, Windows 95/98 version.

Bozorgnia, Y., Campbell K.W., and Niazi, M., 1999, Vertical ground motion: Characteristics, relationship with horizontal component, and building-code implications; Proceedings of the SMIP99 seminar on utilization of strong-motion data, September 15, Oakland, pp. 23-49.

Bryant, W.A., and Hart, E.W., 2007, Fault-rupture hazard zones in California, Alquist-Priolo earthquake fault zoning act with index to earthquake fault zones maps; California Geological Survey, Special Publication 42, interim revision.

Buccola Engineering, Inc., 2015, Tentative parcel map & development plan for Viri Estates, 1 sheet, 20-scale, plot dated May 20.

California Building Standards Commission, 2013, California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2, Based on the 2012 International Building Code, 2013 California Historical Building Code, Title 24, Part 8; 2013 California Existing Building Code, Title 24, Part 10.

California Department of Water Resources, 1993, Division of Safety of Dams, Guidelines for the design and construction of small embankments dams, reprinted January.

California Stormwater Quality Association (CASQA), 2003, Stormwater best management practice handbook, new development and redevelopment, dated January.

City of Oceanside Department of Public Works - Engineering Division, 1991, Engineers design and processing manual, dated February 13 (revised August 26, 1992 and reformatted June 2004).

County of San Diego, Department of Planning and Land Use, 2007, Low impact development (LID) handbook, stormwater management strategies, dated December 31.

GeoSoils, Inc., 2007, Evaluation of existing pavement section, portions of Dixie and Grace Streets, Dixie Street Project, Oceanside, San Diego County, California, W.O. 4396-E-SC, dated May 2.

_____, 2005, Response to review comments, Review memoranda dated January 10 and January 27, 2005, 1914 through 1918 Dixie Street, Oceanside, San Diego County, California, W.O. 4396-A1-SC, dated April 27.

_____, 2004, Preliminary geotechnical evaluation, 1914 through 1918 Dixie Street, Oceanside, San Diego County, California, W.O. 4396-A-SC, dated August 9.

Hydrologic Solutions, StormChamber™ installation brochure, pgs. 1 through 8, undated.

Jennings, C.W., 1994, Fault activity map of California and adjacent areas: California Division of Mines and Geology, Map Sheet No. 6, scale 1:750,000.

Kanare, H.M., 2005, Concrete floors and moisture, Engineering Bulletin 119, Portland Cement Association.

Kennedy, M.P., and Tan, SS., 2005, Geologic map of the Oceanside 30' by 60' quadrangle, California, regional map series, scale 1:100,000, California Geologic Survey and United States Geological Survey, www.conservation.ca.gov/cgs/rghm/rgm/preliminary_geologic_maps.html

Romanoff, M., 1957, Underground corrosion, originally issued April 1.

Seed, 2005, Evaluation and mitigation of soil liquefaction hazard “evaluation of field data and procedures for evaluating the risk of triggering (or inception) of liquefaction”, *in* Geotechnical earthquake engineering; short course, San Diego, California, April 8-9.

Sowers and Sowers, 1979, Unified soil classification system (After U. S. Waterways Experiment Station and ASTM 02487-667) in Introductory Soil Mechanics, New York.

State of California, 2015, Civil Code, Sections 895 et seq.

State of California Department of Transportation, Division of Engineering Services, Materials Engineering, and Testing Services, Corrosion Technology Branch, 2003, Corrosion Guidelines, Version 1.0, dated September.

Tan, S.S., and Giffen, D.G., 1995, Landslide hazards in the northern part of the San Diego Metropolitan area, San Diego County, California, Landslide hazard identification map no. 35, Plate 35A, Department of Conservation, Division of Mines and Geology, DMG Open File Report 95-04.

United States Department of Agriculture, 1953, Stereoscopic aerial photographs, flight line: AXN-9M, photograph nos.: 191 and 192, flight date: April 14.

United States Geological Survey, 2013, U.S. Seismic Design Maps, Earthquake Hazards Program, http://geohazards.usgs.gov/designmaps/us/application.php, version 3.1.0, dated July.

_____, 1997, San Luis Rey quadrangle, San Diego County, California, 7.5 minute series, 1:24,000 scale.

APPENDIX B

UPDATED HISTORICAL SEISMICITY

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*    E Q S E A R C H    *
*                       *
*     Version 3.00      *
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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 6912-A-SC

DATE: 06-11-2015

JOB NAME: SEKHI, LLC

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

SITE COORDINATES:
SITE LATITUDE: 33.2012
SITE LONGITUDE: 117.3623

SEARCH DATES:
START DATE: 1800
END DATE: 2015

SEARCH RADIUS:
62.2 mi
100.1 km

ATTENUATION RELATION: 11) Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Cor.
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 0 Depth Source: A
Basement Depth: 5.00 km Campbell SSR: 0 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

EARTHQUAKE SEARCH RESULTS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.193	VIII	14.3(23.1)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.044	VI	25.1(40.5)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.034	V	31.6(50.8)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.039	V	33.4(53.7)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.057	VI	34.5(55.5)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.031	V	34.5(55.5)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.031	V	34.5(55.5)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.041	V	35.4(57.0)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.052	VI	35.9(57.7)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.028	V	38.3(61.6)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.028	V	38.3(61.7)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.028	V	38.3(61.7)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.028	V	38.3(61.7)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.038	V	42.7(68.8)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.025	V	43.2(69.6)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.076	VII	43.3(69.6)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.025	V	43.3(69.6)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.025	V	43.5(70.1)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.029	V	44.0(70.9)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.027	V	44.1(71.0)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.052	VI	45.1(72.7)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.054	VI	46.3(74.5)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.024	IV	47.4(76.3)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.039	V	49.1(79.1)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.023	IV	50.9(81.9)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.027	V	51.7(83.2)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.027	V	52.6(84.7)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.027	V	53.2(85.6)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.021	IV	53.2(85.7)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.021	IV	53.2(85.7)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.021	IV	53.3(85.8)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.020	IV	53.8(86.6)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.020	IV	55.5(89.3)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.041	V	55.5(89.4)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.067	VI	55.7(89.7)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.019	IV	56.2(90.4)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.020	IV	56.2(90.4)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.019	IV	56.2(90.4)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.020	IV	56.2(90.4)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.022	IV	56.2(90.4)
GSG	33.9530	117.7610	07/29/2008	184215.7	14.0	5.30	0.022	IV	56.7(91.3)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.028	V	59.5(95.7)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.018	IV	59.5(95.8)
GSG	33.9325	117.9172	03/29/2014	040942.3	4.8	5.10	0.019	IV	59.7(96.1)
DMG	33.7830	118.1330	10/02/1933	91017.6	0.0	5.40	0.022	IV	59.9(96.3)
MGI	34.1000	117.3000	07/15/1905	2041 0.0	0.0	5.30	0.020	IV	62.2(100.0)

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-END OF SEARCH- 46 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2015

LENGTH OF SEARCH TIME: 216 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 14.3 MILES (23.1 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.193 g

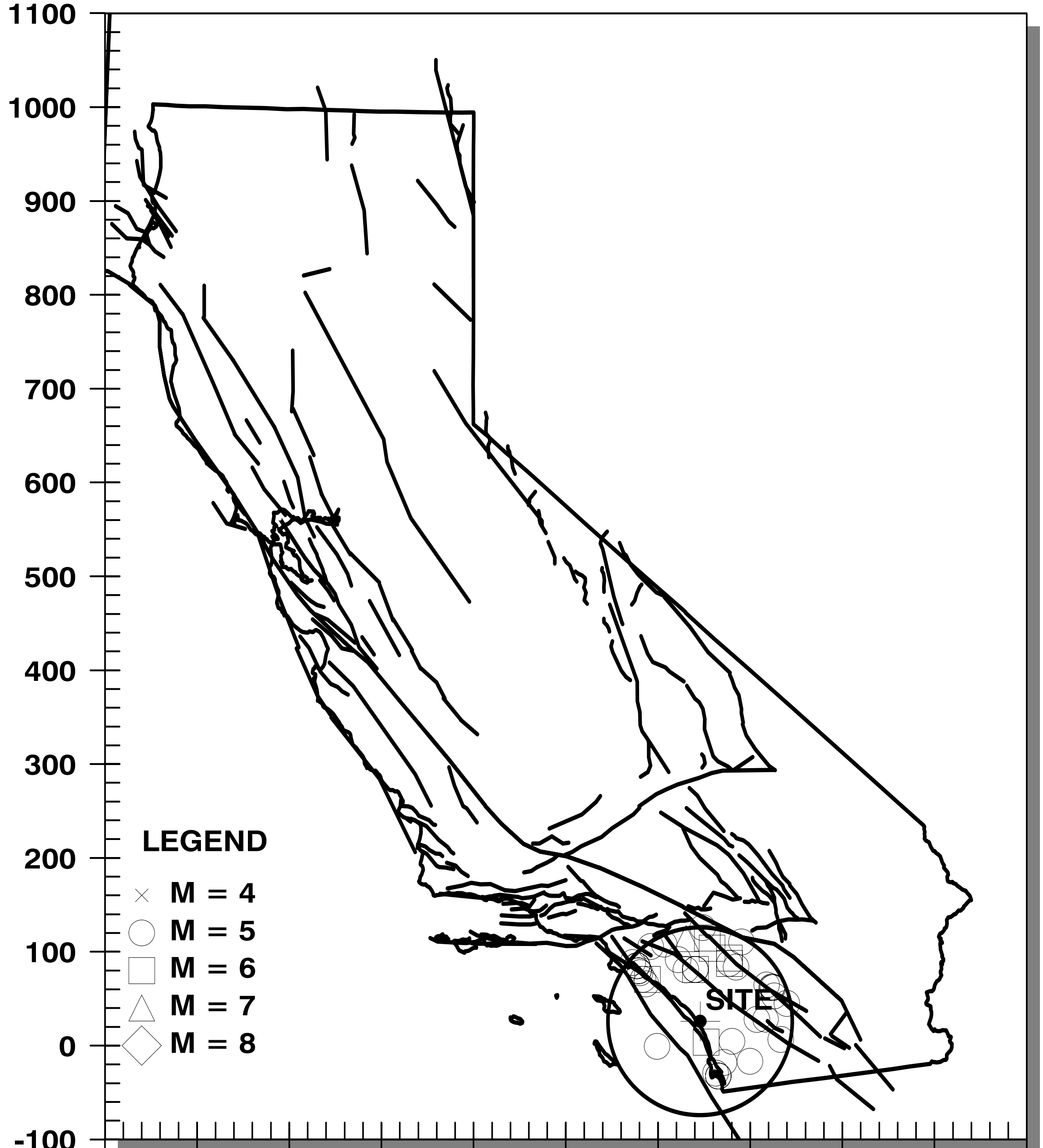
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 1.085
b-value= 0.401
beta-value= 0.924

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	46	0.21395
4.5	46	0.21395
5.0	46	0.21395
5.5	15	0.06977
6.0	8	0.03721
6.5	3	0.01395
7.0	1	0.00465

EARTHQUAKE EPICENTER MAP

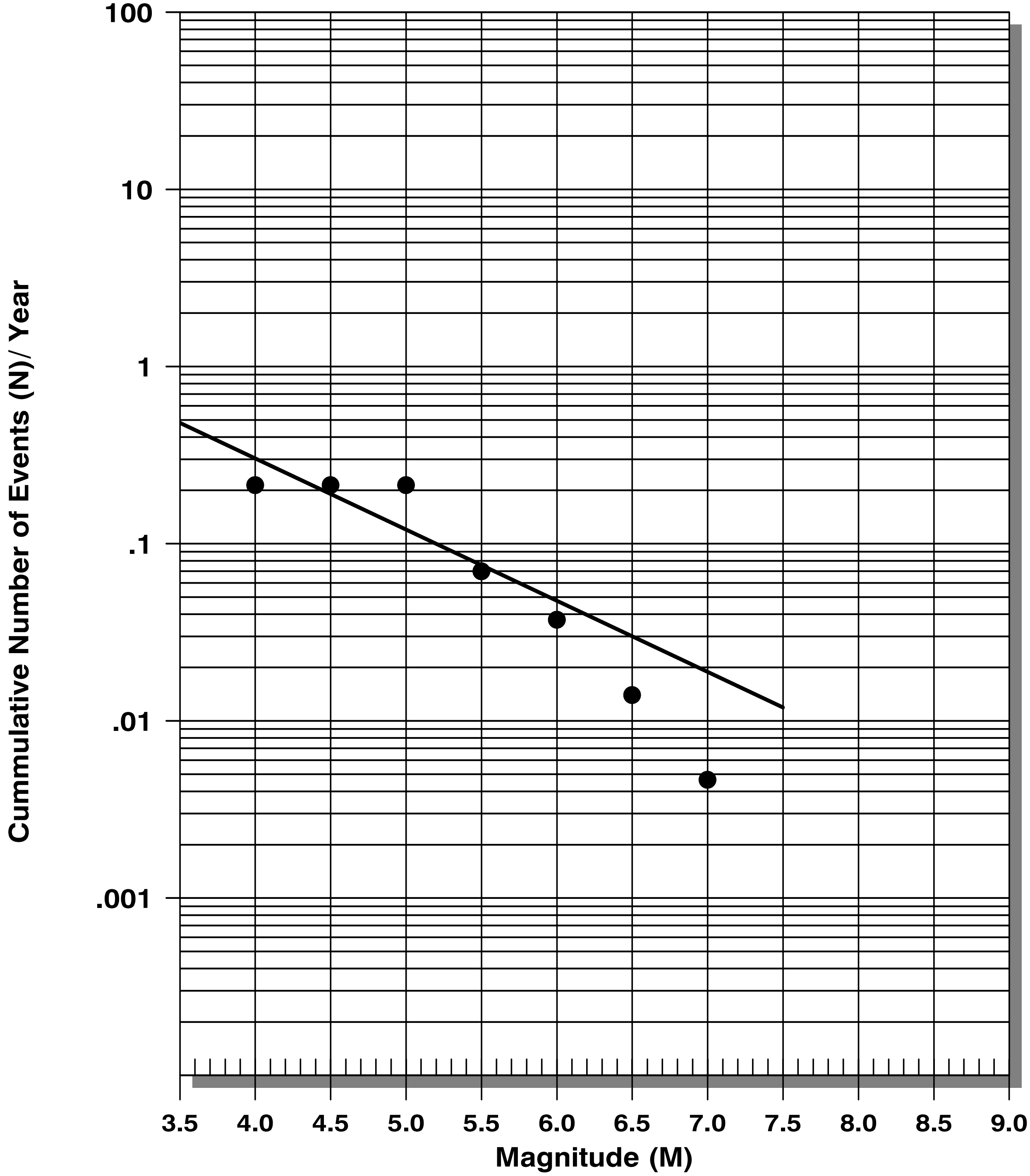
SEKHI, LLC



W.O. 6915-A-SC
PLATE B-4

EARTHQUAKE RECURRENCE CURVE

SEKHI, LLC



W.O. 6915-A-SC
PLATE B-5

APPENDIX C

GENERAL EARTHWORK AND GRADING GUIDELINES

GENERAL EARTHWORK AND GRADING GUIDELINES

General

These guidelines present general procedures and requirements for earthwork and grading as shown on the approved grading plans, including preparation of areas to be filled, placement of fill, installation of subdrains, excavations, and appurtenant structures or flatwork. The recommendations contained in the geotechnical report are part of these earthwork and grading guidelines and would supercede the provisions contained hereafter in the case of conflict. Evaluations performed by the consultant during the course of grading may result in new or revised recommendations which could supercede these guidelines or the recommendations contained in the geotechnical report. Generalized details follow this text.

The contractor is responsible for the satisfactory completion of all earthwork in accordance with provisions of the project plans and specifications and latest adopted code. In the case of conflict, the most onerous provisions shall prevail. The project geotechnical engineer and engineering geologist (geotechnical consultant), and/or their representatives, should provide observation and testing services, and geotechnical consultation during the duration of the project.

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

Prior to the commencement of grading, a qualified geotechnical consultant (soil engineer and engineering geologist) should be employed for the purpose of observing earthwork procedures and testing the fills for general conformance with the recommendations of the geotechnical report(s), the approved grading plans, and applicable grading codes and ordinances.

The geotechnical consultant should provide testing and observation so that an evaluation may be made that the work is being accomplished as specified. It is the responsibility of the contractor to assist the consultants and keep them apprised of anticipated work schedules and changes, so that they may schedule their personnel accordingly.

All remedial removals, clean-outs, prepared ground to receive fill, key excavations, and subdrain installation should be observed and documented by the geotechnical consultant prior to placing any fill. It is the contractor's responsibility to notify the geotechnical consultant when such areas are ready for observation.

Laboratory and Field Tests

Maximum dry density tests to determine the degree of compaction should be performed in accordance with American Standard Testing Materials test method ASTM designation D-1557. Random or representative field compaction tests should be performed in accordance with test methods ASTM designation D-1556, D-2937 or D-2922, and D-3017, at intervals of approximately ±2 feet of fill height or approximately every 1,000 cubic yards placed. These criteria would vary depending on the soil conditions and the size of the project. The location and frequency of testing would be at the discretion of the geotechnical consultant.

Contractor's Responsibility

All clearing, site preparation, and earthwork performed on the project should be conducted by the contractor, with observation by a geotechnical consultant, and staged approval by the governing agencies, as applicable. It is the contractor's responsibility to prepare the ground surface to receive the fill, to the satisfaction of the geotechnical consultant, and to place, spread, moisture condition, mix, and compact the fill in accordance with the recommendations of the geotechnical consultant. The contractor should also remove all non-earth material considered unsatisfactory by the geotechnical consultant.

Notwithstanding the services provided by the geotechnical consultant, it is the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the earthwork in strict accordance with applicable grading guidelines, latest adopted codes or agency ordinances, geotechnical report(s), and approved grading plans. Sufficient watering apparatus and compaction equipment should be provided by the contractor with due consideration for the fill material, rate of placement, and climatic conditions. If, in the opinion of the geotechnical consultant, unsatisfactory conditions such as questionable weather, excessive oversized rock or deleterious material, insufficient support equipment, etc., are resulting in a quality of work that is not acceptable, the consultant will inform the contractor, and the contractor is expected to rectify the conditions, and if necessary, stop work until conditions are satisfactory.

During construction, the contractor shall properly grade all surfaces to maintain good drainage and prevent ponding of water. The contractor shall take remedial measures to control surface water and to prevent erosion of graded areas until such time as permanent drainage and erosion control measures have been installed.

SITE PREPARATION

All major vegetation, including brush, trees, thick grasses, organic debris, and other deleterious material, should be removed and disposed of off-site. These removals must be concluded prior to placing fill. In-place existing fill, soil, alluvium, colluvium, or rock materials, as evaluated by the geotechnical consultant as being unsuitable, should be removed prior to any fill placement. Depending upon the soil conditions, these materials may be reused as compacted fills. Any materials incorporated as part of the compacted fills should be approved by the geotechnical consultant.

Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, or other structures not located prior to grading, are to be removed or treated in a manner recommended by the geotechnical consultant. Soft, dry, spongy, highly fractured, or otherwise unsuitable ground, extending to such a depth that surface processing cannot adequately improve the condition, should be overexcavated down to firm ground and approved by the geotechnical consultant before compaction and filling operations continue. Overexcavated and processed soils, which have been properly mixed and moisture conditioned, should be re-compacted to the minimum relative compaction as specified in these guidelines.

Existing ground, which is determined to be satisfactory for support of the fills, should be scarified (ripped) to a minimum depth of 6 to 8 inches, or as directed by the geotechnical consultant. After the scarified ground is brought to optimum moisture content, or greater and mixed, the materials should be compacted as specified herein. If the scarified zone is greater than 6 to 8 inches in depth, it may be necessary to remove the excess and place the material in lifts restricted to about 6 to 8 inches in compacted thickness.

Existing ground which is not satisfactory to support compacted fill should be overexcavated as required in the geotechnical report, or by the on-site geotechnical consultant. Scarification, disc harrowing, or other acceptable forms of mixing should continue until the soils are broken down and free of large lumps or clods, until the working surface is reasonably uniform and free from ruts, hollows, hummocks, mounds, or other uneven features, which would inhibit compaction as described previously.

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical [h:v]), the ground should be stepped or benched. The lowest bench, which will act as a key, should be a minimum of 15 feet wide and should be at least 2 feet deep into firm material, and approved by the geotechnical consultant. In fill-over-cut slope conditions, the recommended minimum width of the lowest bench or key is also 15 feet, with the key founded on firm material, as designated by the geotechnical consultant. As a general rule, unless specifically recommended otherwise by the geotechnical consultant, the minimum width of fill keys should be equal to ½ the height of the slope.

Standard benching is generally 4 feet (minimum) vertically, exposing firm, acceptable material. Benching may be used to remove unsuitable materials, although it is understood that the vertical height of the bench may exceed 4 feet. Pre-stripping may be considered for unsuitable materials in excess of 4 feet in thickness.

All areas to receive fill, including processed areas, removal areas, and the toes of fill benches, should be observed and approved by the geotechnical consultant prior to placement of fill. Fills may then be properly placed and compacted until design grades (elevations) are attained.

COMPACTED FILLS

Any earth materials imported or excavated on the property may be utilized in the fill provided that each material has been evaluated to be suitable by the geotechnical consultant. These materials should be free of roots, tree branches, other organic matter, or other deleterious materials. All unsuitable materials should be removed from the fill as directed by the geotechnical consultant. Soils of poor gradation, undesirable expansion potential, or substandard strength characteristics may be designated by the consultant as unsuitable and may require blending with other soils to serve as a satisfactory fill material.

Fill materials derived from benching operations should be dispersed throughout the fill area and blended with other approved material. Benching operations should not result in the benched material being placed only within a single equipment width away from the fill/bedrock contact.

Oversized materials defined as rock, or other irreducible materials, with a maximum dimension greater than 12 inches, should not be buried or placed in fills unless the location of materials and disposal methods are specifically approved by the geotechnical consultant. Oversized material should be taken offsite, or placed in accordance with recommendations of the geotechnical consultant in areas designated as suitable for rock disposal. GSI anticipates that soils to be utilized as fill material for the subject project may contain some rock. Appropriately, the need for rock disposal may be necessary during grading operations on the site. From a geotechnical standpoint, the depth of any rocks, rock fills, or rock blankets, should be a sufficient distance from finish grade. This depth is generally the same as any overexcavation due to cut-fill transitions in hard rock areas, and generally facilitates the excavation of structural footings and substructures. Should deeper excavations be proposed (i.e., deepened footings, utility trenching, swimming pools, spas, etc.), the developer may consider increasing the hold-down depth of any rocky fills to be placed, as appropriate. In addition, some agencies/jurisdictions mandate a specific hold-down depth for oversize materials placed in fills. The hold-down depth, and potential to encounter oversize rock, both within fills, and occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet, unless specified differently in the text of this report. The governing agency may require that these materials need to be deeper, crushed, or reduced to less than 12 inches in maximum dimension, at their discretion.

To facilitate future trenching, rock (or oversized material), should not be placed within the hold-down depth feet from finish grade, the range of foundation excavations, future utilities, or underground construction unless specifically approved by the governing agency, the geotechnical consultant, and/or the developer's representative.

If import material is required for grading, representative samples of the materials to be utilized as compacted fill should be analyzed in the laboratory by the geotechnical consultant to evaluate it's physical properties and suitability for use onsite. Such testing should be performed three (3) days prior to importation. If any material other than that previously tested is encountered during grading, an appropriate analysis of this material should be conducted by the geotechnical consultant as soon as possible.

Approved fill material should be placed in areas prepared to receive fill in near horizontal layers, that when compacted, should not exceed about 6 to 8 inches in thickness. The geotechnical consultant may approve thick lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness. Each layer should be spread evenly and blended to attain uniformity of material and moisture suitable for compaction.

Fill layers at a moisture content less than optimum should be watered and mixed, and wet fill layers should be aerated by scarification, or should be blended with drier material. Moisture conditioning, blending, and mixing of the fill layer should continue until the fill materials have a uniform moisture content at, or above, optimum moisture.

After each layer has been evenly spread, moisture conditioned, and mixed, it should be uniformly compacted to a minimum of 90 percent of the maximum density as evaluated by ASTM test designation D 1557, or as otherwise recommended by the geotechnical consultant. Compaction equipment should be adequately sized and should be specifically designed for soil compaction, or of proven reliability to efficiently achieve the specified degree of compaction.

Where tests indicate that the density of any layer of fill, or portion thereof, is below the required relative compaction, or improper moisture is in evidence, the particular layer or portion shall be re-worked until the required density and/or moisture content has been attained. No additional fill shall be placed in an area until the last placed lift of fill has been tested and found to meet the density and moisture requirements, and is approved by the geotechnical consultant.

In general, per the latest adopted version of the California Building Code (CBC), fill slopes should be designed and constructed at a gradient of 2:1 (h:v), or flatter. Compaction of slopes should be accomplished by over-building a minimum of 3 feet horizontally, and subsequently trimming back to the design slope configuration. Testing shall be performed as the fill is elevated to evaluate compaction as the fill core is being developed. Special efforts may be necessary to attain the specified compaction in the fill slope zone. Final slope shaping should be performed by trimming and removing loose materials with appropriate equipment. A final evaluation of fill slope compaction should be based on observation and/or testing of the finished slope face. Where compacted fill slopes are designed steeper than 2:1 (h:v), prior approval from the governing agency, specific material types, a higher minimum relative compaction, special reinforcement, and special grading procedures will be recommended.

If an alternative to over-building and cutting back the compacted fill slopes is selected, then special effort should be made to achieve the required compaction in the outer 10 feet of each lift of fill by undertaking the following:

1. An extra piece of equipment consisting of a heavy, short-shanked sheepsfoot should be used to roll (horizontal) parallel to the slopes continuously as fill is placed. The sheepsfoot roller should also be used to roll perpendicular to the

slopes, and extend out over the slope to provide adequate compaction to the face of the slope.

2. Loose fill should not be spilled out over the face of the slope as each lift is compacted. Any loose fill spilled over a previously completed slope face should be trimmed off or be subject to re-rolling.

3. Field compaction tests will be made in the outer (horizontal) ±2 to ±8 feet of the slope at appropriate vertical intervals, subsequent to compaction operations.

4. After completion of the slope, the slope face should be shaped with a small tractor and then re-rolled with a sheepsfoot to achieve compaction to near the slope face. Subsequent to testing to evaluate compaction, the slopes should be grid-rolled to achieve compaction to the slope face. Final testing should be used to evaluate compaction after grid rolling.

5. Where testing indicates less than adequate compaction, the contractor will be responsible to rip, water, mix, and recompact the slope material as necessary to achieve compaction. Additional testing should be performed to evaluate compaction.

SUBDRAIN INSTALLATION

Subdrains should be installed in approved ground in accordance with the approximate alignment and details indicated by the geotechnical consultant. Subdrain locations or materials should not be changed or modified without approval of the geotechnical consultant. The geotechnical consultant may recommend and direct changes in subdrain line, grade, and drain material in the field, pending exposed conditions. The location of constructed subdrains, especially the outlets, should be recorded/surveyed by the project civil engineer. Drainage at the subdrain outlets should be provided by the project civil engineer.

EXCAVATIONS

Excavations and cut slopes should be examined during grading by the geotechnical consultant. If directed by the geotechnical consultant, further excavations or overexcavation and refilling of cut areas should be performed, and/or remedial grading of cut slopes should be performed. When fill-over-cut slopes are to be graded, unless otherwise approved, the cut portion of the slope should be observed by the geotechnical consultant prior to placement of materials for construction of the fill portion of the slope. The geotechnical consultant should observe all cut slopes, and should be notified by the contractor when excavation of cut slopes commence.

If, during the course of grading, unforeseen adverse or potentially adverse geologic conditions are encountered, the geotechnical consultant should investigate, evaluate, and make appropriate recommendations for mitigation of these conditions. The need for cut slope buttressing or stabilizing should be based on in-grading evaluation by the geotechnical consultant, whether anticipated or not.

Unless otherwise specified in geotechnical and geological report(s), no cut slopes should be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies. Additionally, short-term stability of temporary cut slopes is the contractor's responsibility.

Erosion control and drainage devices should be designed by the project civil engineer and should be constructed in compliance with the ordinances of the controlling governmental agencies, and/or in accordance with the recommendations of the geotechnical consultant.

COMPLETION

Observation, testing, and consultation by the geotechnical consultant should be conducted during the grading operations in order to state an opinion that all cut and fill areas are graded in accordance with the approved project specifications. After completion of grading, and after the geotechnical consultant has finished observations of the work, final reports should be submitted, and may be subject to review by the controlling governmental agencies. No further excavation or filling should be undertaken without prior notification of the geotechnical consultant or approved plans.

All finished cut and fill slopes should be protected from erosion and/or be planted in accordance with the project specifications and/or as recommended by a landscape architect. Such protection and/or planning should be undertaken as soon as practical after completion of grading.

PRELIMINARY OUTDOOR POOL/SPA DESIGN RECOMMENDATIONS

The following preliminary recommendations are provided for consideration in pool/spa design and planning. Actual recommendations should be provided by a qualified geotechnical consultant, based on site specific geotechnical conditions, including a subsurface investigation, differential settlement potential, expansive and corrosive soil potential, proximity of the proposed pool/spa to any slopes with regard to slope creep and lateral fill extension, as well as slope setbacks per Code, and geometry of the proposed improvements. Recommendations for pools/spas and/or deck flatwork underlain by expansive soils, or for areas with differential settlement greater than ¼-inch over 40 feet horizontally, will be more onerous than the preliminary recommendations presented below.

The 1:1 (h:v) influence zone of any nearby retaining wall site structures should be delineated on the project civil drawings with the pool/spa. This 1:1 (h:v) zone is defined as a plane up from the lower-most heel of the retaining structure, to the daylight grade of the nearby building pad or slope. If pools/spas or associated pool/spa improvements are constructed within this zone, they should be re-positioned (horizontally or vertically) so that they are supported by earth materials that are outside or below this 1:1 plane. If this is not possible given the area of the building pad, the owner should consider eliminating these improvements or allow for increased potential for lateral/vertical deformations and associated distress that may render these improvements unusable in the future, unless they are periodically repaired and maintained. The conditions and recommendations presented herein should be disclosed to all owners and any interested/affected parties.

General

1. The equivalent fluid pressure to be used for the pool/spa design should be 60 pounds per cubic foot (pcf) for pool/spa walls with level backfill, and 75 pcf for a 2:1 sloped backfill condition. In addition, backdrains should be provided behind pool/spa walls subjacent to slopes.

2. Passive earth pressure may be computed as an equivalent fluid having a density of 150 pcf, to a maximum lateral earth pressure of 1,000 pounds per square foot (psf).

3. An allowable coefficient of friction between soil and concrete of 0.30 may be used with the dead load forces.

4. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

5. Where pools/spas are planned near structures, appropriate surcharge loads need to be incorporated into design and construction by the pool/spa designer. This includes, but is not limited to landscape berms, decorative walls, footings, built-in barbeques, utility poles, etc.

6. All pool/spa walls should be designed as “free standing” and be capable of supporting the water in the pool/spa without soil support. The shape of pool/spa in cross section and plan view may affect the performance of the pool, from a geotechnical standpoint. Pools and spas should also be designed in accordance with the latest adopted Code. Minimally, the bottoms of the pools/spas, should maintain a distance H/3, where H is the height of the slope (in feet), from the slope face. This distance should not be less than 7 feet, nor need not be greater than 40 feet.

7. The soil beneath the pool/spa bottom should be uniformly moist with the same stiffness throughout. If a fill/cut transition occurs beneath the pool/spa bottom, the cut portion should be overexcavated to a minimum depth of 48 inches, and replaced with compacted fill, such that there is a uniform blanket that is a minimum of 48 inches below the pool/spa shell. If very low expansive soil is used for fill, the

fill should be placed at a minimum of 95 percent relative compaction, at optimum moisture conditions. This requirement should be 90 percent relative compaction at over optimum moisture if the pool/spa is constructed within or near expansive soils. The potential for grading and/or re-grading of the pool/spa bottom, and attendant potential for shoring and/or slot excavation, needs to be considered during all aspects of pool/spa planning, design, and construction.

8. If the pool/spa is founded entirely in compacted fill placed during rough grading, the deepest portion of the pool/spa should correspond with the thickest fill on the lot.

9. Hydrostatic pressure relief valves should be incorporated into the pool and spa designs. A pool/spa under-drain system is also recommended, with an appropriate outlet for discharge.

10. All fittings and pipe joints, particularly fittings in the side of the pool or spa, should be properly sealed to prevent water from leaking into the adjacent soils materials, and be fitted with slip or expandible joints between connections transecting varying soil conditions.

11. An elastic expansion joint (flexible waterproof sealant) should be installed to prevent water from seeping into the soil at all deck joints.

12. A reinforced grade beam should be placed around skimmer inlets to provide support and mitigate cracking around the skimmer face.

13. In order to reduce unsightly cracking, deck slabs should minimally be 4 inches thick, and reinforced with No. 3 reinforcing bars at 18 inches on-center. All slab reinforcement should be supported to ensure proper mid-slab positioning during the placement of concrete. Wire mesh reinforcing is specifically not recommended. Deck slabs should not be tied to the pool/spa structure. Pre-moistening and/or pre-soaking of the slab subgrade is recommended, to a depth of 12 inches (optimum moisture content), or 18 inches (120 percent of the soil's optimum moisture content, or 3 percent over optimum moisture content, whichever is greater), for very low to low, and medium expansive soils, respectively. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks. Slab underlayment should consist of a 1- to 2-inch leveling course of sand (S.E.>30) and a minimum of 4 to 6 inches of Class 2 base compacted to 90 percent. Deck slabs within the H/3 zone, where H is the height of the slope (in feet), will have an increased potential for distress relative to other areas outside of the H/3 zone. If distress is undesirable, improvements, deck slabs or flatwork should not be constructed closer than H/3 or 7 feet (whichever is greater) from the slope face, in order to reduce, but not eliminate, this potential.

14. Pool/spa bottom or deck slabs should be founded entirely on competent bedrock, or properly compacted fill. Fill should be compacted to achieve a minimum 90 percent relative compaction, as discussed above. Prior to pouring concrete, subgrade soils below the pool/spa decking should be throughly watered to achieve a moisture content that is at least 2 percent above optimum moisture content, to a depth of at least 18 inches below the bottom of slabs. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks.

15. In order to reduce unsightly cracking, the outer edges of pool/spa decking to be bordered by landscaping, and the edges immediately adjacent to the pool/spa, should be underlain by an 8-inch wide concrete cutoff shoulder (thickened edge) extending to a depth of at least 12 inches below the bottoms of the slabs to mitigate excessive infiltration of water under the pool/spa deck. These thickened edges should be reinforced with two No. 4 bars, one at the top and one at the bottom. Deck slabs may be minimally reinforced with No. 3 reinforcing bars placed at 18 inches on-center, in both directions. All slab reinforcement should be supported on chairs to ensure proper mid-slab positioning during the placement of concrete.

16. Surface and shrinkage cracking of the finish slab may be reduced if a low slump and water-cement ratio are maintained during concrete placement. Concrete utilized should have a minimum compressive strength of 4,000 psi. Excessive water added to concrete prior to placement is likely to cause shrinkage cracking, and should be avoided. Some concrete shrinkage cracking, however, is unavoidable.

17. Joint and sawcut locations for the pool/spa deck should be determined by the design engineer and/or contractor. However, spacings should not exceed 6 feet on center.

18. Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees), should be anticipated. All excavations should be observed by a representative of the geotechnical consultant, including the project geologist and/or geotechnical engineer, prior to workers entering the excavation or trench, and minimally conform to Cal/OSHA ("Type C" soils may be assumed), state, and local safety codes. Should adverse conditions exist, appropriate recommendations should be offered at that time by the geotechnical consultant. GSI does not consult in the area of safety engineering and the safety of the construction crew is the responsibility of the pool/spa builder.

19. It is imperative that adequate provisions for surface drainage are incorporated by the owners into their overall improvement scheme. Ponding water, ground saturation and flow over slope faces, are all situations which must be avoided to enhance long term performance of the pool/spa and associated improvements, and reduce the likelihood of distress.

20. Regardless of the methods employed, once the pool/spa is filled with water, should it be emptied, there exists some potential that if emptied, significant distress may occur. Accordingly, once filled, the pool/spa should not be emptied unless evaluated by the geotechnical consultant and the pool/spa builder.

21. For pools/spas built within (all or part) of the Code setback and/or geotechnical setback, as indicated in the site geotechnical documents, special foundations are recommended to mitigate the affects of creep, lateral fill extension, expansive soils and settlement on the proposed pool/spa. Most municipalities or County reviewers do not consider these effects in pool/spa plan approvals. As such, where pools/spas are proposed on 20 feet or more of fill, medium or highly expansive soils, or rock fill with limited "cap soils" and built within Code setbacks, or within the influence of the creep zone, or lateral fill extension, the following should be considered during design and construction:

    OPTION A: Shallow foundations with or without overexcavation of the pool/spa "shell," such that the pool/spa is surrounded by 5 feet of very low to low expansive soils (without irreducible particles greater that 6 inches), and the pool/spa walls closer to the slope(s) are designed to be free standing. GSI recommends a pool/spa under-drain or blanket system (see attached Typical Pool/Spa Detail). The pool/spa builders and owner in this optional construction technique should be generally satisfied with pool/spa performance under this scenario; however, some settlement, tilting, cracking, and leakage of the pool/spa is likely over the life of the project.

    OPTION B: Pier supported pool/spa foundations with or without overexcavation of the pool/spa shell such that the pool/spa is surrounded by 5 feet of very low to low expansive soils (without irreducible particles greater than 6 inches), and the pool/spa walls closer to the slope(s) are designed to be free standing. The need for a pool/spa under-drain system may be installed for leak detection purposes. Piers that support the pool/spa should be a minimum of 12 inches in diameter and at a spacing to provide vertical and lateral support of the pool/spa, in accordance with the pool/spa designers recommendations current applicable Codes. The pool/spa builder and owner in this second scenario construction technique should be more satisfied with pool/spa performance. This construction will reduce settlement and creep effects on the pool/spa; however, it will not eliminate these potentials, nor make the pool/spa "leak-free."

22. The temperature of the water lines for spas and pools may affect the corrosion properties of site soils, thus, a corrosion specialist should be retained to review all spa and pool plans, and provide mitigative recommendations, as warranted. Concrete mix design should be reviewed by a qualified corrosion consultant and materials engineer.

23. All pool/spa utility trenches should be compacted to 90 percent of the laboratory standard, under the full-time observation and testing of a qualified geotechnical consultant. Utility trench bottoms should be sloped away from the primary structure on the property (typically the residence).

24. Pool and spa utility lines should not cross the primary structure's utility lines (i.e., not stacked, or sharing of trenches, etc.).

25. The pool/spa or associated utilities should not intercept, interrupt, or otherwise adversely impact any area drain, roof drain, or other drainage conveyances. If it is necessary to modify, move, or disrupt existing area drains, subdrains, or tightlines, then the design civil engineer should be consulted, and mitigative measures provided. Such measures should be further reviewed and approved by the geotechnical consultant, prior to proceeding with any further construction.

26. The geotechnical consultant should review and approve all aspects of pool/spa and flatwork design prior to construction. A design civil engineer should review all aspects of such design, including drainage and setback conditions. Prior to acceptance of the pool/spa construction, the project builder, geotechnical consultant and civil designer should evaluate the performance of the area drains and other site drainage pipes, following pool/spa construction.

27. All aspects of construction should be reviewed and approved by the geotechnical consultant, including during excavation, prior to the placement of any additional fill, prior to the placement of any reinforcement or pouring of any concrete.

28. Any changes in design or location of the pool/spa should be reviewed and approved by the geotechnical and design civil engineer prior to construction. Field adjustments should not be allowed until written approval of the proposed field changes are obtained from the geotechnical and design civil engineer.

29. Disclosure should be made to owners and builders, contractors, and any interested/affected parties, that pools/spas built within about 15 feet of the top of a slope, and/or H/3, where H is the height of the slope (in feet), <u>will</u> experience some movement or tilting. While the pool/spa shell or coping may not necessarily crack, the levelness of the pool/spa will likely tilt toward the slope, and may not be esthetically pleasing. The same is true with decking, flatwork and other improvements in this zone.

30. Failure to adhere to the above recommendations will significantly increase the potential for distress to the pool/spa, flatwork, etc.

31. Local seismicity and/or the design earthquake will cause some distress to the pool/spa and decking or flatwork, possibly including total functional and economic loss.

32. The information and recommendations discussed above should be provided to any contractors and/or subcontractors, or owners, interested/affected parties, etc., that may perform or may be affected by such work.

JOB SAFETY

General

At GSI, getting the job done safely is of primary concern. The following is the company's safety considerations for use by all employees on multi-employer construction sites. On-ground personnel are at highest risk of injury, and possible fatality, on grading and construction projects. GSI recognizes that construction activities will vary on each site, and that site safety is the prime responsibility of the contractor; however, everyone must be safety conscious and responsible at all times. To achieve our goal of avoiding accidents, cooperation between the client, the contractor, and GSI personnel must be maintained.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of field personnel on grading and construction projects:

Safety Meetings: GSI field personnel are directed to attend contractor's regularly scheduled and documented safety meetings.

Safety Vests: Safety vests are provided for, and are to be worn by GSI personnel, at all times, when they are working in the field.

Safety Flags: Two safety flags are provided to GSI 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.

Flashing Lights: All vehicles stationary in the grading area shall use rotating or flashing amber beacons, or strobe lights, on the vehicle during all field testing. While operating a vehicle in the grading area, the emergency flasher on the vehicle shall be activated.

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. A primary concern should be the technician's safety. Efforts will be made to coordinate locations with the grading contractor's authorized representative, and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractor's authorized representative (supervisor, grade checker, dump man, operator, etc.) should direct excavation of the pit and safety during the test period. Of paramount concern should be the soil technician's safety, and obtaining enough tests to represent the fill.

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

A zone of non-encroachment should be established for all test pits. No grading equipment should enter this zone during the testing procedure. The zone should extend approximately 50 feet outward from the center of the test pit. This zone is established for safety and to avoid excessive ground vibration, which typically decreases test results.

When taking slope tests, the technician should park the vehicle directly above or below the test location. If this is not possible, a prominent flag should be placed at the top of the slope. The contractor's representative should effectively keep all equipment at a safe operational distance (e.g., 50 feet) away from the slope during this 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, well away from the equipment traffic pattern. The contractor should inform our personnel of all changes to haul roads, cut and fill areas or other factors that may affect site access and site safety.

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 required, by company policy, to immediately withdraw and notify his/her supervisor. The grading contractor's representative will be contacted in an effort to affect a solution. However, in the interim, no further testing will be performed until the situation is rectified. Any fill placed 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 the technician's attention and notify this office. Effective communication and coordination between the contractor's representative and the soil technician is strongly encouraged in order to implement the above safety plan.

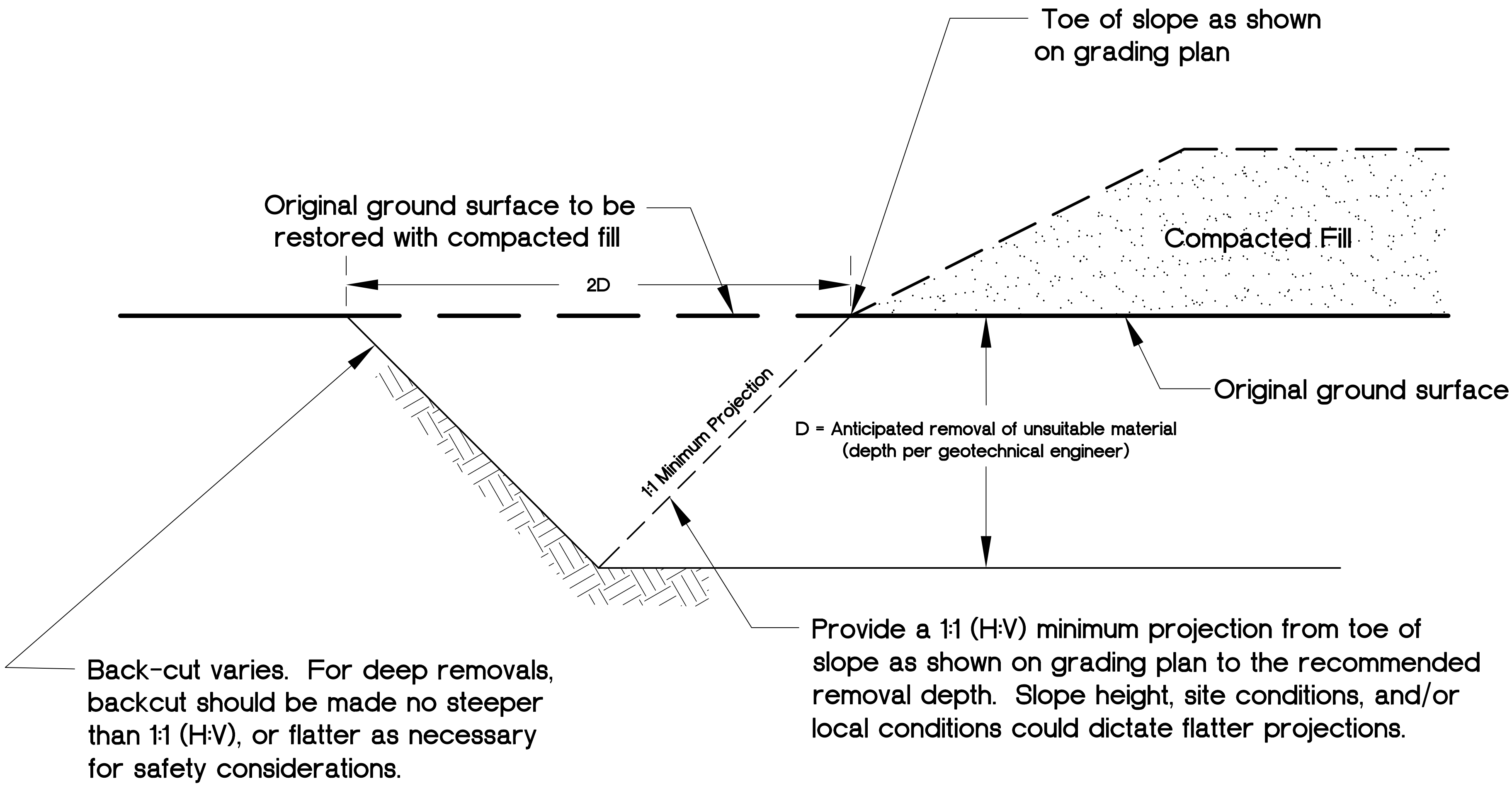
Trench and Vertical Excavation

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or other debris which could fall into the trench; or 3) displays any other evidence of any unsafe conditions regardless of depth.

All trench excavations or vertical cuts in excess of 5 feet deep, which any person enters, should be shored or laid back. Trench access should be provided in accordance with Cal/OSHA and/or state and local standards. Our personnel are directed not to enter any trench by being lowered or “riding down” on the equipment.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor’s representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns or other reasons could be subject to reprocessing and/or removal.

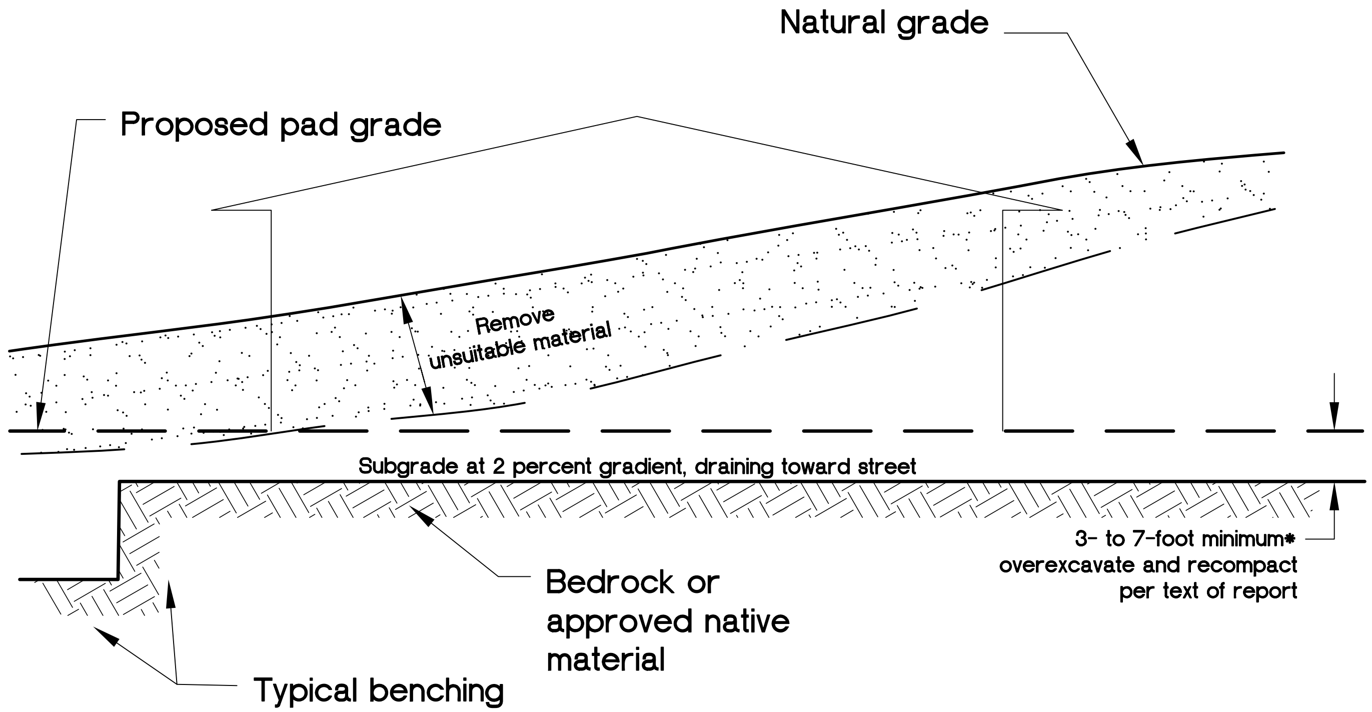
If GSI personnel become aware of anyone working beneath an unsafe trench wall or vertical excavation, we have a legal obligation to put the contractor and owner/developer on notice to immediately correct the situation. If corrective steps are not taken, GSI then has an obligation to notify Cal/OSHA and/or the proper controlling authorities.



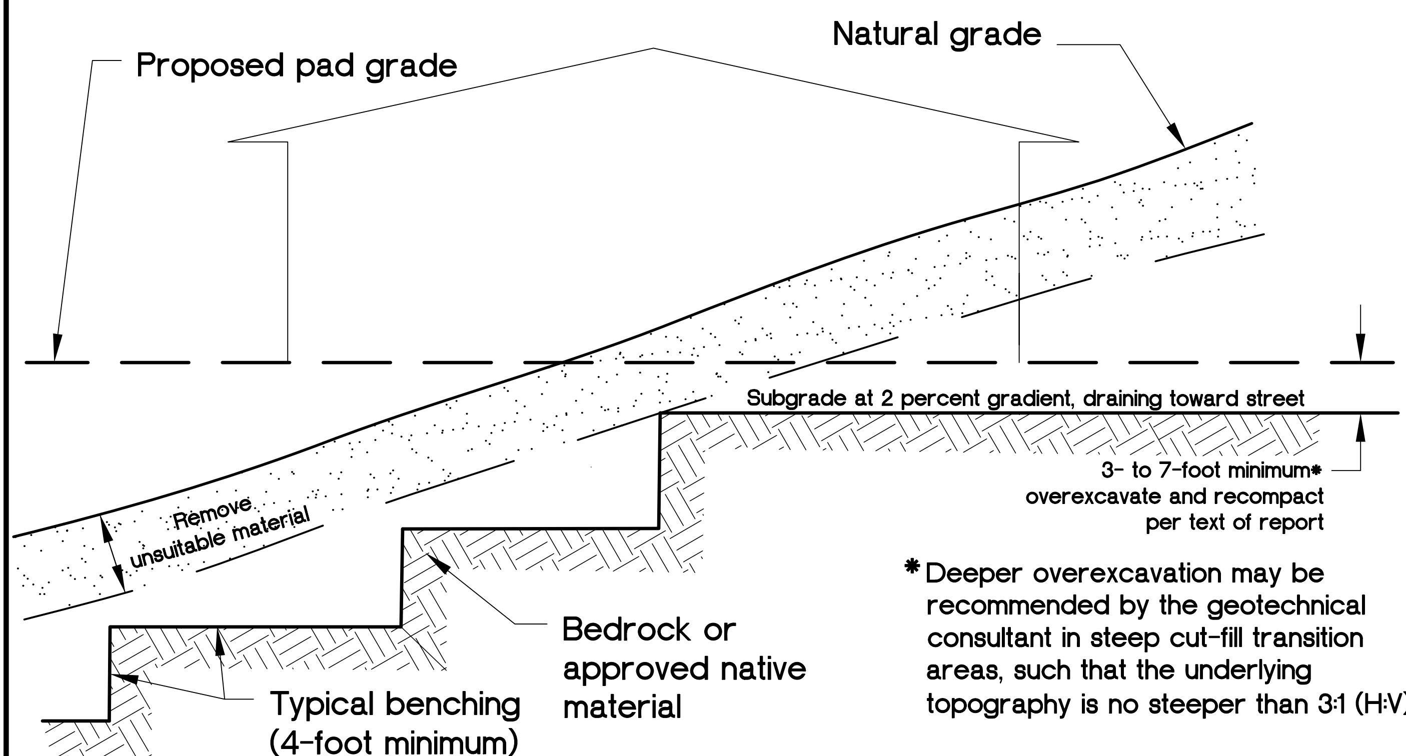
GeoSoils, Inc.

FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL

Plate C-3

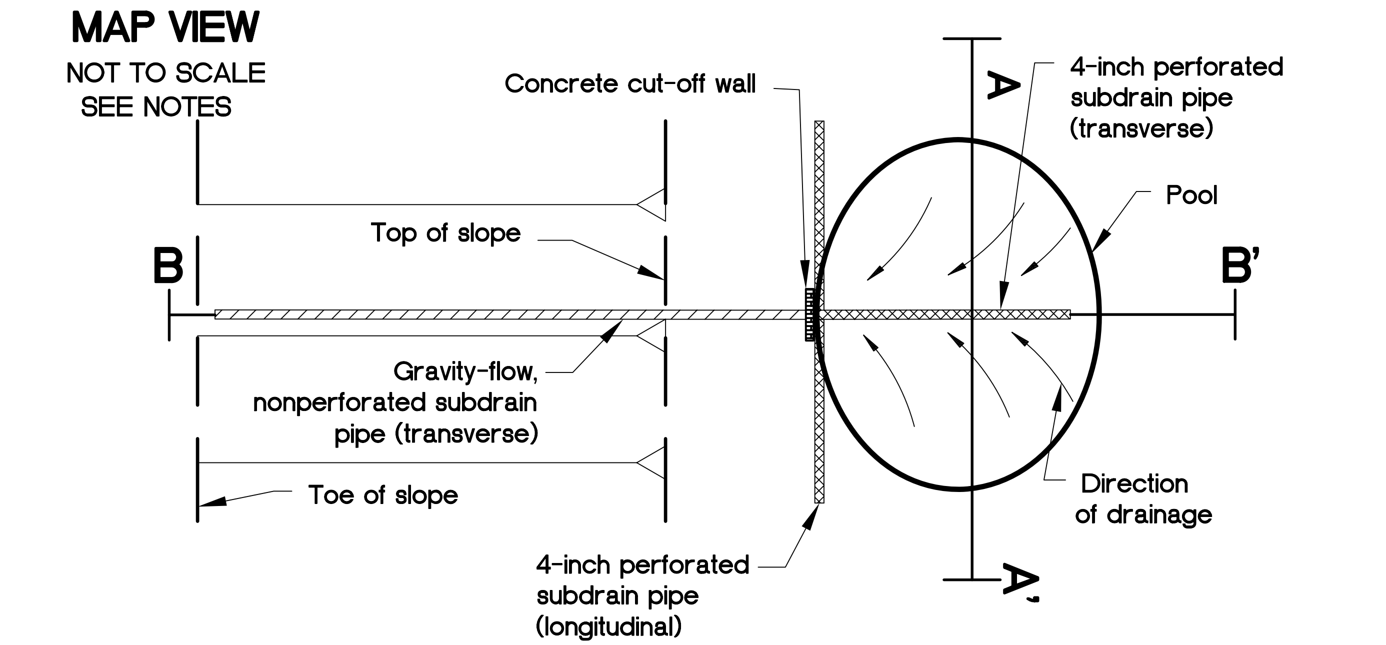


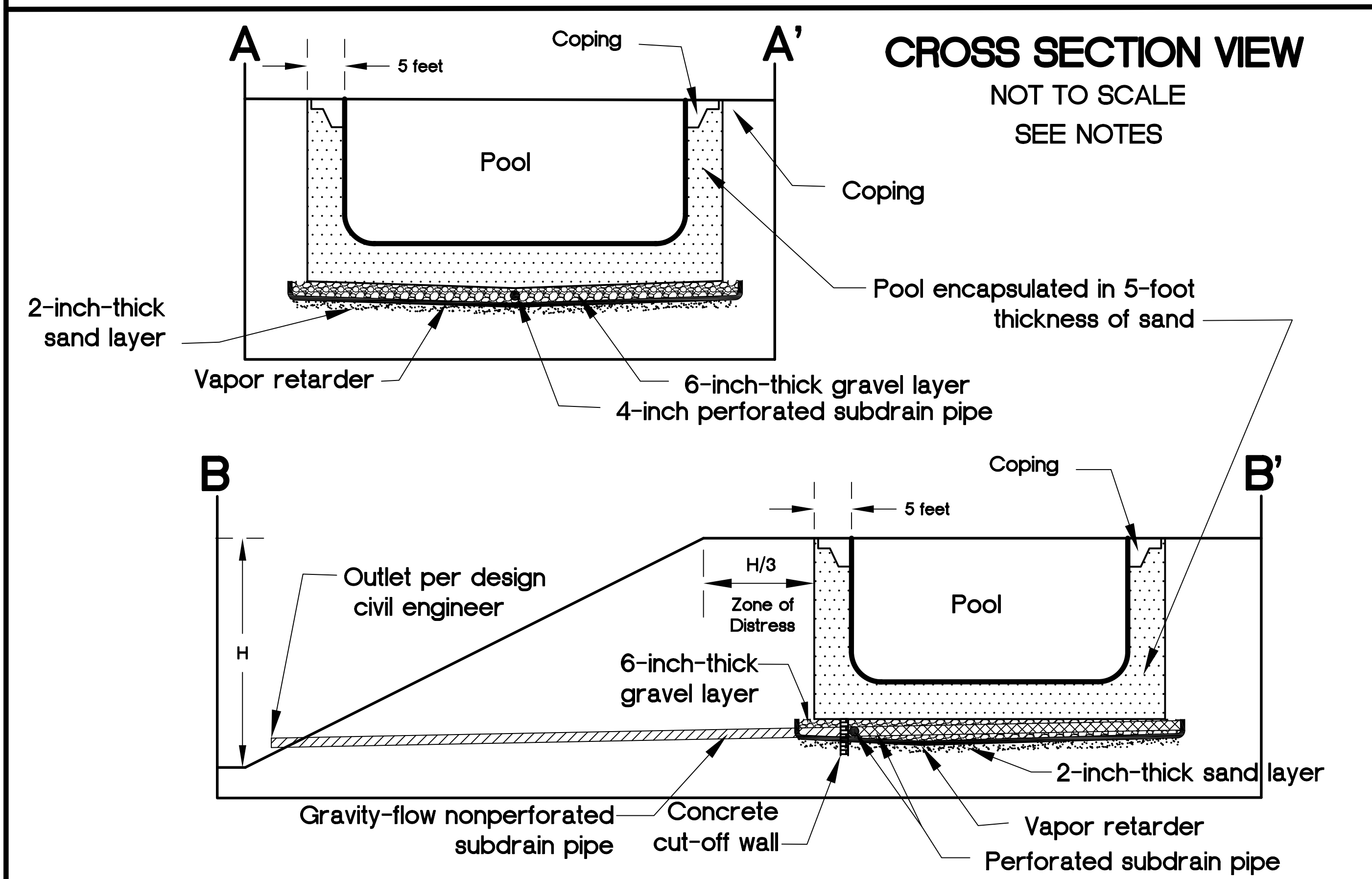
CUT LOT OR MATERIAL-TYPE TRANSITION



CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.	TRANSITION LOT DETAILS	Plate C-12



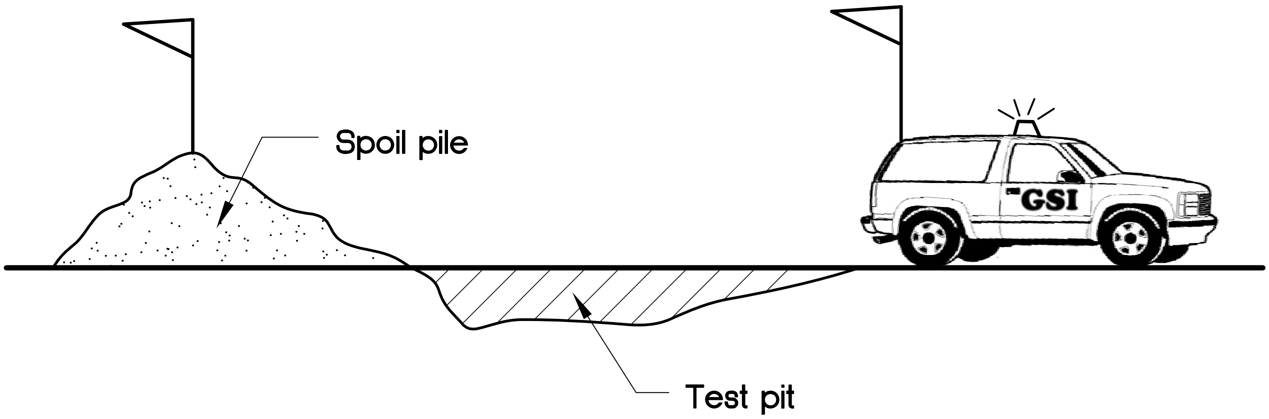


NOTES:

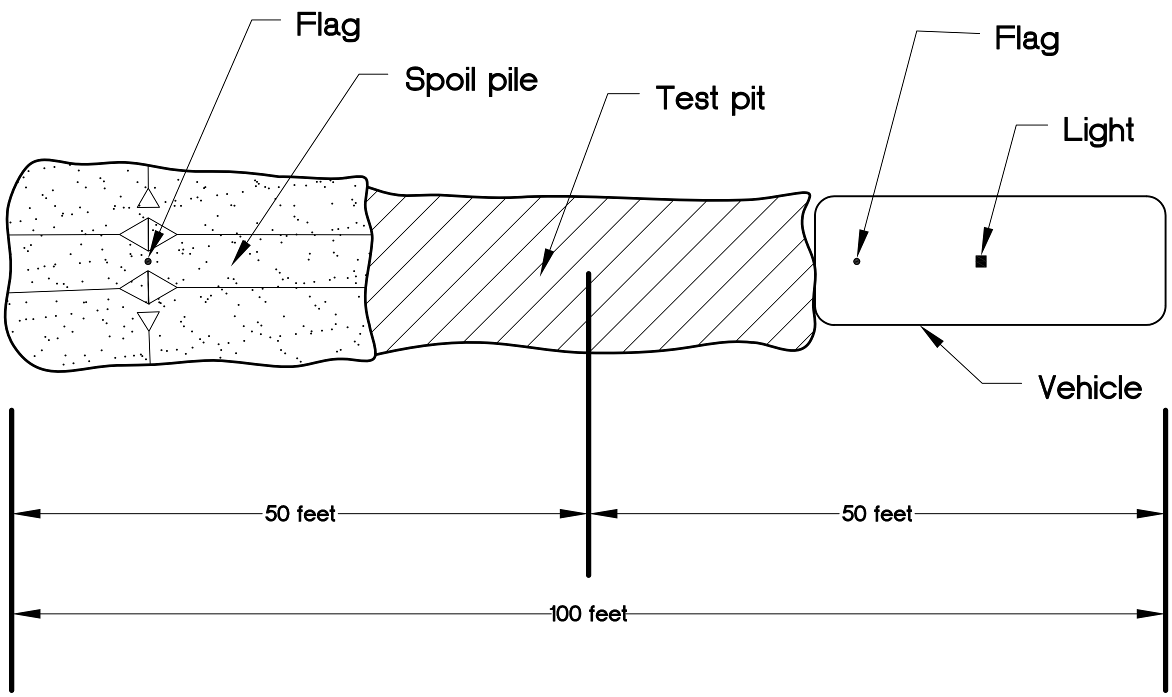
1. 6-inch-thick, clean gravel (¾ to 1½ inch) sub-base encapsulated in Mirafi 140N or equivalent, underlain by a 15-mil vapor retarder, with 4-inch-diameter perforated pipe longitudinal connected to 4-inch-diameter perforated pipe transverse. Connect transverse pipe to 4-inch-diameter nonperforated pipe at low point and outlet or to sump pump area.
2. Pools on fills thicker than 20 feet should be constructed on deep foundations; otherwise, distress (tilting, cracking, etc.) should be expected.
3. Design does not apply to infinity-edge pools/spas.

GeoSoils, Inc.	TYPICAL POOL/SPA DETAIL	Plate C-17

SIDE VIEW



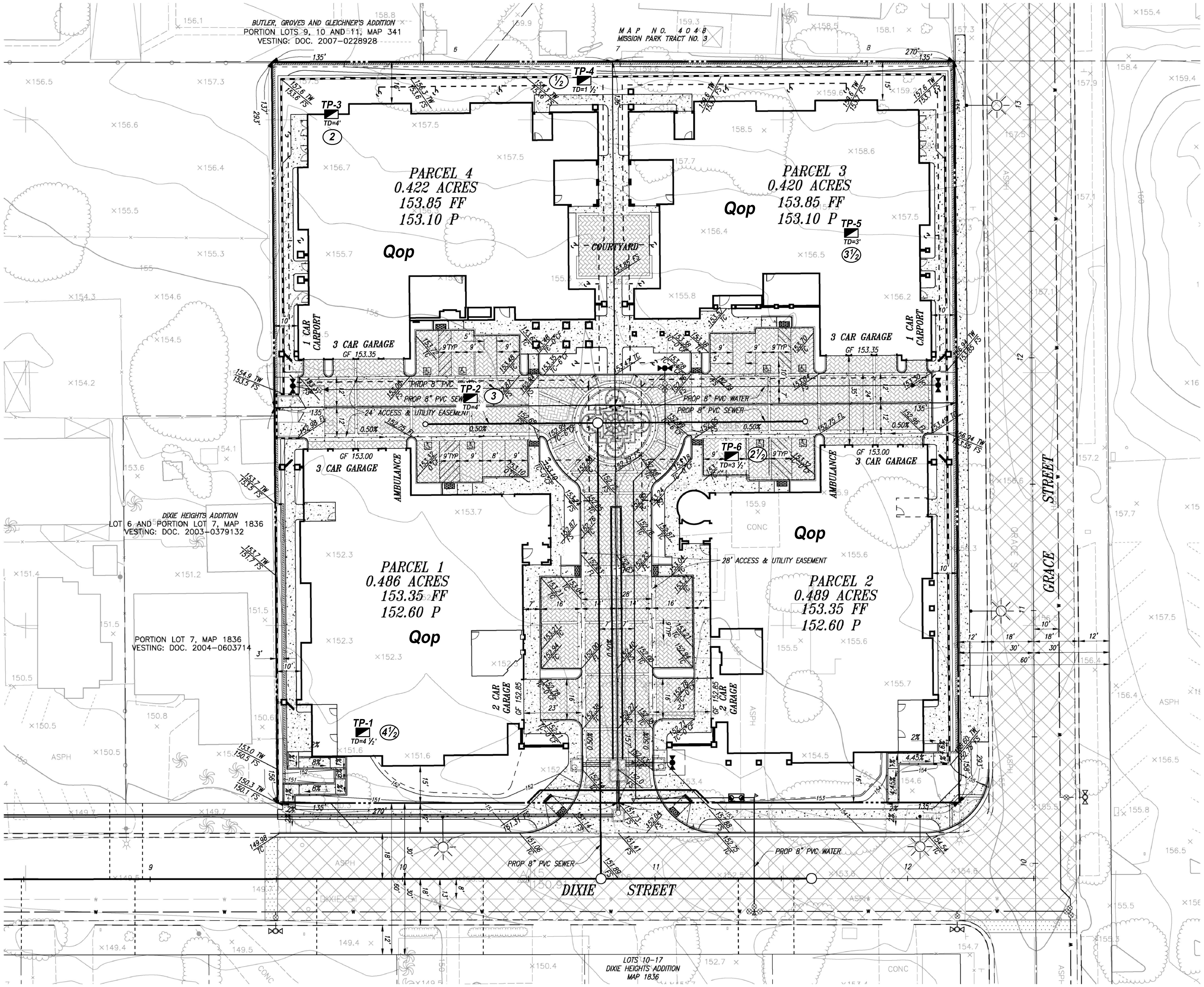
TOP VIEW

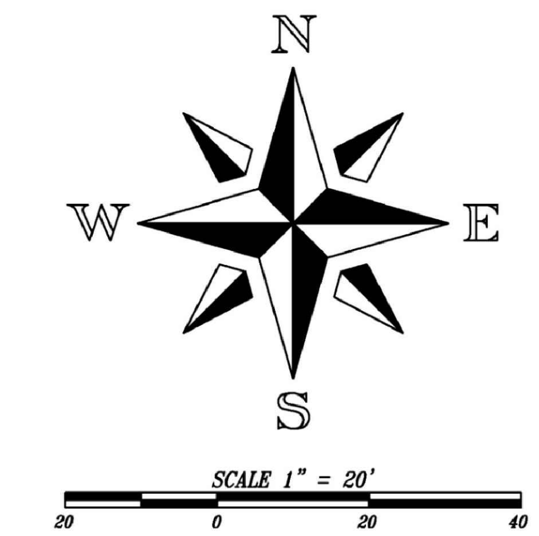


GeoSoils, Inc.	TEST PIT SAFETY DIAGRAM	Plate C-20

TENTATIVE PARCEL MAP & DEVELOPMENT PLAN
FOR
VIRI ESTATES

GPA15-00000
Z15-00000
C15-00000
D15-00000
P15-00000





GSI LEGEND

Qop	QUATERNARY OLD PARALIC DEPOSITS
TP-6 TD=4½'	APPROXIMATE LOCATION OF EXPLORATORY TEST PIT (GSI, 2004)
4½	APPROXIMATE DEPTH IN FEET TO SUITABLE OLD PARALIC DEPOSITS BELOW EXISTING GRADE

ALL LOCATIONS ARE APPROXIMATE

This document or efile is not a part of the Construction Documents and should not be relied upon as being an accurate depiction of design.

BASE MAP
PREPARED IN THE OFFICE OF:

BUCCOLA ENGINEERING, inc
760/721-2000
3142 Vista Way, Suite 301, Oceanside, CA 92056

PHILIP D. BUCCOLA RCE 27732 DATE

GeoSoils, Inc.

TEST PIT LOCATION MAP

Plate 1

W.O. 6912-A-SC	DATE: 06/15	SCALE: 1" = 20'