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April 8, 2022
P/W 2202-04
Report No. 2202-04-B-2

Attention: **Don Ross**

Subject: **Due Diligence Geotechnical Report for Cota Vera Swim Club, Parcel C, Otay Ranch Village 8 West, City of Chula Vista, California**

References: See Appendix A

Gentlepersons,

Presented herein is Advanced Geotechnical Solutions, Inc.'s, (AGS) geotechnical Due Diligence Report based upon the *Otay Swim Club – Site Context Plan* (prepared by Brightview Design Group) on Parcel C of the Otay Village 8 West Development, City of Chula Vista, California. The purpose of this evaluation is to provide *Home Fed Corporation* with geologic and geotechnical information that should be considered in your due diligence process towards development of the subject property. In preparing this report, AGS has observed the existing site conditions and reviewed the referenced geotechnical reports and geologic maps. The subject parcel is currently being mass graded and has not achieved the final mass graded condition. Accordingly, the proposed due diligence investigation will be based upon assumed conditions that will be verified at a later date once the lot achieves its interim sheet graded configuration. AGS has been retained by Otay Land Company to provide geotechnical and geologic services during mass grading. At the conclusion of mass grading, AGS will prepare an as-graded compaction report summarizing our observations and testing.

In this document, we first describe the site and the proposed development, and summarize the anticipated soils and geologic conditions. Secondly, we assess key geologic/geotechnical issues which could potentially impact site development and outline possible measures which can be employed to mitigate and or minimize the impact these issues could have on the development as currently proposed. Finally, we provide conclusions regarding the feasibility of site development and provide preliminary design recommendations. This abbreviated document is intended to serve as a "first step" for acquiring geotechnical information useful for planning and design.

1.0 INTRODUCTION

1.1. Existing Site Conditions

The irregular shaped property is located north of the intersection of La Media Parkway and Avenida Caprise within the Otay Ranch Village 8 West master development in the City of Chula Vista, California. As currently proposed the interim sheet graded condition will vary from a high of approximately 416 msl to a low of 410 msl with sheet flow down to the south. Based on the current Phase 2 - Mass Grading Plans, a temporary basin is proposed in the southerly portion of the site adjacent to La Media Parkway and Avenida Caprise. A temporary storm drain line has been partially constructed in the easterly portion of the site for drainage of the temporary basin.

A majority of the earthwork related to the mass grading has been completed at this time. The site is predominantly underlain by Otay Formation consisting of brown to light gray siltstone/sandstone/gritstone. Several stockpiles of soil and construction debris are located throughout the project and minor undocumented fill veneers are anticipated to exist locally across the site.

1.2. Proposed Development

Preliminary grading plans were unavailable for our review at this time; however, it is anticipated that minor cuts and fills generally on the order of 5 feet or less will be required to achieve the precise graded condition. The provided conceptual site plan indicates the site will be developed to support two pools, a single-story office building with an attached covered patio, a single-story bathroom, pool equipment and storage building, a covered yoga area, and associated improvements including underground utilities, paved parking areas, and hardscape. It is anticipated that the structures will have concrete masonry unit and wood-frame construction and be supported by shallow slab-on-grade foundations.

2.0 SITE INVESTIGATION

2.1. Previous Studies

Several geotechnical studies have been performed onsite prior to the initiation of the mass grading of the site (see Appendix A-References). Representatives of AGS have been involved with the initial design since early 2003. Currently, AGS has been retained to provide geotechnical and geologic testing services associated with the ongoing mass grading. At the conclusion of mass grading, AGS will prepare a final compaction report summarizing our observations and testing during grading and prepare a geologic map of the *as-graded* conditions on the lot.

2.2. Current Investigation

In preparing this *Geotechnical Due Diligence Investigation* representatives of AGS conducted a site walkover, reviewed the current site plan, and transferred the approximate “as-graded” geologic contacts, removal bottoms and subdrain locations onto the site plan included herewith as Plate 1.

3.0 REGIONAL AND SITE GEOLOGY

The subject site is situated within the Peninsular Ranges Geomorphic Province. The Peninsular Ranges province occupies the southwestern portion of California and extends southward to the southern tip of Baja California. In general, the province consists of young, steeply sloped, northwest trending mountain ranges underlain by metamorphosed Late Jurassic to Early Cretaceous-aged extrusive volcanic rock and Cretaceous-aged igneous plutonic rock of the Peninsular Ranges Batholith. The westernmost portion of the province is predominantly underlain by younger marine and non-marine sedimentary rocks. The Peninsular Ranges’ dominant structural feature is northwest-southeast trending crustal blocks bounded by active faults of the San Andreas transform system.

3.1. Site Geology

Prior to grading, the site was mantled by thin deposits of topsoil and alluvium and a rock stockpile overlying Otay Formation. The surficial soils were removed during grading to expose competent Otay Formation. The Otay Formation (map symbol Oo) generally consists of brown to light gray siltstone/sandstone/gritstone with variable clay content. The Otay formation is generally poorly to moderately well indurated and is locally cross-bedded. The approximate distribution of geologic units is shown on Plate 1.

3.2. Groundwater

Based on our review of previous geotechnical reports in the area and our observations during ongoing mass grading, groundwater was not encountered during excavations within the site boundaries. A roughly north to south trending tributary drainage with intermittent flows transected the northeasterly corner of the site prior to site grading. A canyon subdrain was installed at the bottom of the drainage and the area was backfilled with compacted artificial fill.

The San Diego Reservoir is located offsite to the north of the Swim Club. During grading of the design cut slope underlying the reservoir site, seepage was observed. As a mitigation measure, the cut slope was replaced by a compacted fill buttress with two keyway drains to intercept the seepage.

It should be noted that a detention basin is proposed in the southern portion of the site. Nuisance seepage related to impounded water may be encountered in nearby excavations; however, it is anticipated that this would be a transient condition. If encountered, recommendations can be provided on an as-needed basis. Groundwater is not anticipated to adversely impact site development.

3.3. Subsurface Drainage

There are no existing subsurface drainage systems beneath the project site and the need for additional subsurface drainage is not anticipated. A partially constructed storm drain line is located beneath the easterly portion of the site. It is our understanding the storm drain line is intended to convey storm water from the proposed temporary basin and will be removed at a later date.

3.4. Flooding

According to available FEMA maps, the site is not in a FEMA identified flood hazard area.

3.5. Subsidence/Ground Fissuring

Due to the presence of competent Otay Formation materials, the potential for subsidence/settlement and ground fissuring is unlikely.

3.6. Landsliding/Slope Instability

Given the relatively flat gradients across the site and the competent nature of fill and formational materials in the site vicinity, the potential for landsliding, mass wasting, and/or surficial instability onsite is considered to be remote.

4.0 SEISMIC HAZARDS

The project is in the tectonically active southern California and will likely experience some effects from future earthquakes. There are no Alquist-Priolo Fault Hazard Zones mapped within the subject property. The State of California Seismic Hazards Mapping program identifying areas of potential liquefaction and earthquake induced landsliding has not addressed the Otay Quadrangle as of this writing.

The type or severity of seismic hazards affecting the site is chiefly dependent upon the distance to and direction from causative faults, the intensity and duration of the seismic events, and the onsite soil characteristics. The seismic hazard may be primary, such as surface rupture and/or ground shaking, or secondary, such as liquefaction or landsliding.

The following is a site-specific discussion of ground motion parameters, earthquake-induced landslide hazards, settlement, and liquefaction. The purpose of this analysis is to identify potential seismic hazards and propose mitigations, if necessary, to reduce the hazard to an acceptable level of risk. The following seismic hazards discussion is guided by the 2019 California Building Code, CDMG (2008), and Martin and Lew (1998).

4.1. Surface Fault Rupture

Active, potentially active and inactive faults are not known to exist at the site. According to the literature, the nearest known active fault is the Rose Canyon Fault located approximately 10.2 miles (16.4 km) west of the project site. The potentially active La Nacion Fault is located approximately 2 miles west of the site. Accordingly, the potential for fault surface rupture within the project is not significant.

4.2. Seismicity

As noted, the site is within the tectonically active southern California area and is approximately 10.2 miles from the active Rose Canyon Fault. The potential exists for strong ground motion that may affect future improvements. At this point in time, non-critical structures (commercial, residential, and industrial) are usually designed according to the 2019 California Building Code and that of the controlling local agency.

4.3. Liquefaction

Seismic agitation of relatively loose saturated sands and silty sands can result in a buildup of pore pressure. If the pore pressure exceeds the overburden stresses, a temporary quick condition known as liquefaction may occur. Within the project, the potential for liquefaction in both the pre- and post- development condition is “very low” due to the lack of liquefaction susceptible earth materials, the dense nature of the onsite geologic units, and the lack of shallow groundwater.

4.4. Dynamic Settlement

Dynamic settlement occurs in response to an earthquake event affecting loose sandy earth materials. The potential of dynamic settlement at the subject site is anticipated to be “very low” due to the presence of competent formational materials.

4.5. Seismically Induced Landsliding

Given the relatively flat topography onsite the potential for landsliding is negligible.

4.6. Tsunami

Our review of the 2009 Tsunami Inundation Map for Emergency Planning prepared by CalEMA, indicates that the site is not within the tsunami inundation limits.

5.0 SEISMIC DESIGN PARAMETERS

Based on our previous subsurface exploration and testing during mass grading, the site has been classified as Site Class C consisting of a very dense soil and soft rock profile due to the presence of Otay Formation. Table 5.0 presents seismic design parameters in accordance with 2019 California Building Code (CBC) and mapped spectral acceleration parameters (United States Geological Survey, 2022) utilizing site coordinates of Latitude 32.5992°N and Longitude 116.9751°W.

TABLE 5.0
2019 CBC SEISMIC DESIGN PARAMETERS (SITE CLASS C)

Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s	0.754
Mapped Spectral Acceleration Parameter at Period 1-Second, S_1	0.275
Site Coefficient, F_a	1.200
Site Coefficient, F_v	1.500
Adjusted MCE_R[1] Spectral Response Acceleration Parameter at Short Period, S_{MS}	0.904
1-Second Period Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1}	0.412
Short Period Design Spectral Response Acceleration Parameter, S_{DS}	0.603
1-Second Period Design Spectral Response Acceleration Parameter, S_{D1}	0.275
Peak Ground Acceleration, PGA_M[2]	0.393
Seismic Design Category	D

Notes:
[1] Risk-Targeted Maximum Considered Earthquake
[2] Peak Ground Acceleration adjusted for site effects

6.0 PRELIMINARY EARTHWORK RECOMMENDATIONS

Based on the available information, the proposed improvements are considered feasible from a geotechnical standpoint. The conclusions and recommendations presented herein are preliminary and are suitable for initial design and budgeting of the proposed project. Grading plans, when available, should be reviewed by AGS. Based on that review, modifications to the recommendations may be warranted.

6.1. <u>Earthwork Recommendations and Considerations</u>

All grading should be accomplished under the observation and testing of the project soils engineer and engineering geologist or their authorized representative in accordance with the recommendations contained in the approved geotechnical reports, the Grading Specifications provided by AGS (2018), the project specifications, and the 2019 California Building Code. Prior to fill placement, the bottoms of all removal areas should be observed and approved by the engineering geologist/soils engineer or their authorized representative. Onsite materials are suitable for use as compacted fill provided any deleterious materials, including organic materials, are removed. Mixing and moisture control of materials will be necessary. All fills should be compacted at least 90 percent of the maximum dry density as determined by ASTM D1557. Fill should be placed in thin (6 to 8-inch) lifts, moisture conditioned to optimum moisture or slightly above, and compacted to 90 percent of the maximum dry density (ASTM D1557) until the desired grade is achieved.

Import soils, if required, should consist of clean, structural quality, compactable materials similar to the on-site soils and should be free of trash, debris or other objectionable materials. Import soils should be tested and approved by the Geotechnical Consultant prior to importing.

6.1.1. Site Preparation

Existing vegetation, trash, debris, and other deleterious materials should be removed and wasted from the site prior to commencing removal of unsuitable soils and placement of compacted fill materials.

6.1.2. Removals

Materials that have been disturbed during the ongoing construction activities should be removed in their entirety prior to placement of compacted engineered fill. Weathered and saturated artificial fill materials, if present, should also be removed. The removed soils may be reused as fill provided they are clean of debris and vegetation, including roots.

6.1.3. Overexcavation for Building Pads

It is recommended that overexcavation be performed on cut/fill transition lots and lots underlain by hard/resistant formational materials. Overexcavation should maintain a minimum one (1) percent gradient to the front of the lot or deep fill area. General overexcavation recommendations are provided below. Specific recommendations can be provided at the conclusion of mass grading and when precise grading plans become available.

6.1.3.1. Transition Lots

Where design or remedial grading activities create a cut/fill transition in engineered fill and formational materials, overexcavation of the cut and shallow fill portions should be performed such that at least three (3) to four (4) feet of compacted fill exist over the pad. If a steep cut/fill transition occurs, consideration should be given to deepening the lot overexcavations to achieve a minimum underlying fill depth of H/3 (where H is the maximum depth of fill within affected lots).

6.1.3.2. Hard Rock/Formation Lots

It is recommended that lots underlain by hard rock or resistant formational materials be overexcavated during site grading to facilitate foundation and shallow trench excavations. Overexcavation to a minimum depth of three (3) feet below pad grade or one (1) foot below bottom of footing elevation, whichever is deeper, is recommended. Replacement fill should be eight (8) inch minus in particle size and compacted to project specifications.

6.1.4. Overexcavation for Retaining Walls

Retaining wall footings should be supported entirely on compacted fill *or* formational materials. The need for overexcavation below retaining wall foundations areas should be based on exposed conditions and/or the depth wall foundation elements. Minimally, the depth of undercut should be at least one (1) foot below bottom of foundation elements. The final determination of foundation undercut depths should be established upon review of detailed approved wall plans and final as-graded condition.

If overexcavation for walls is not attainable, cut/fill conditions could exist. In consideration that retaining wall foundation would be supported on dissimilar materials, AGS recommends that expansion joints be constructed at the transition between the two materials. These joints are proposed to help mitigate the potential for distress related to differential wall movement at the transition locations. The methodology utilized to construct expansion joints should be addressed and approved by the wall designer.

6.1.5. Fill Material/Import Soils

Most of the soils encountered during our subsurface exploration should be suitable for use as engineered fill provided that they are free of any debris and vegetation, including roots, and any rocks that are greater than 6 inches in diameter.

Any imported fill material should consist of granular soil having a “very low” to “low” expansion potential (expansion index of 50 or less). Import should also have a low corrosion potential (chloride content less than 500 parts per million, soluble sulfate content of less than 0.1 percent, and pH of 5.5 or higher). Potential import material to be used as fill should be evaluated by AGS prior to importing or using as fill. At least three working days should be provided to the geotechnical consultant to sample and test the potential import material.

6.1.6. Compaction

Prior to placement of compacted fill, the exposed excavation bottoms should be observed by AGS. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 8 inches and watered or dried, as needed, to achieve a moisture content approximately 2 percent above optimum moisture content. The scarified bottom should then be compacted to 90 percent relative compaction in accordance with the latest version of ASTM D 1557 test method.

Fill materials should be moisture conditioned to approximately 2 percent above optimum moisture content prior to placement. All fill, except where stated otherwise, should be mechanically compacted in layers not more than 8 inches thick to at least 90 percent of the maximum laboratory density for the materials used. Successive lifts should be placed in a like manner until the desired finished grades are achieved. A representative of AGS should be onsite to observe the fill placement operation and perform compaction testing.

6.1.7. Utility Trench Excavation and Backfill

All utility trenches should be shored or laid back in accordance with applicable OSHA standards. Excavations in bedrock areas should be made in consideration of underlying geologic structure. The geotechnical consultant should be consulted on these issues during construction.

Trench backfill should be compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557. Onsite soils will not be suitable for use as bedding material but will be suitable for use in backfill, provided oversized materials are removed. No surcharge loads should be imposed above excavations. This includes spoil piles, lumber, concrete trucks or other construction materials and equipment. Drainage above excavations should be directed away from the banks. Care should be taken to avoid saturation of the soils. Compaction should be accomplished by mechanical means. Jetting of native soils will not be acceptable.

To reduce moisture penetration beneath the slab-on-grade areas, shallow utility trenches should be backfilled with lean concrete or concrete slurry where they intercept the foundation perimeter. As an alternative, such excavations can be backfilled with native soils, moisture-conditioned to over optimum, and compacted to a minimum of 90 percent relative compaction.

6.2. Excavations and Temporary Backcuts

Where possible, temporary unsurcharged excavations should be sloped no steeper than 1:1 (horizontal to vertical) inclination in formational materials and 1½:1 (H:V) for compacted fill which correspond to CalOSHA Type B and Type C soils, respectively.

The top of sloped excavations should be barricaded so that vehicles and storage loads do not encroach within 10 feet of the excavations. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. AGS should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended along the tops of the slopes in order to prevent runoff water from entering the excavation and eroding the slope faces.

AGS representatives should observe the excavations so that any necessary modifications based on variations in the encountered soil conditions can be made. All applicable safety requirements and regulations, including CalOSHA requirements, should be met.

6.3. Preliminary Pavement Design

For preliminary design and estimating purposes the following flexible (asphalt concrete) pavement structural sections presented in Table 6.3 can be used for the range of likely traffic indices. The structural sections are based on an assumed R-Value of 20. Final pavement design recommendations should be based on sampling and testing of post-grading conditions and will be provided to the City of Chula Vista for review and approval.

TABLE 6.3
PRELIMINARY ASPHALT CONCRETE PAVEMENT SECTIONS

Traffic Index (TI)	Asphaltic Concrete (AC) (inch)	Aggregate Base (AB) (inch)
4.5	3	6
5.0	3	7
6.0	4	8.5

Subgrade soils should be compacted to at least 95 percent of maximum density as determined by ASTM D-1557. Aggregate base materials should be compacted to at least 95 percent of maximum density as determined by California Test 216.

7.0 PRELIMINARY DESIGN RECOMMENDATIONS

7.1. Structural Design

Structural engineering plans are not currently available. It is expected that for typical one-story structures and loading conditions (1 to 3 ksf for spread and continuous footings), that conventional slab-on-grade foundations will be utilized. Post-tensioned slab-on-grade foundations are also suitable for the proposed structures. Preliminary foundation design recommendations are presented the following sections.

It is anticipated that the as-graded near-surface soils will predominantly vary from “low” to “medium” in expansion potential when tested in general accordance with ASTM D 4829. Localized near-surface soils may exhibit “high” expansion potential. Upon the completion of precise grading, finish grade samples should be collected and tested to develop specific recommendations as they relate to final foundation design recommendations for individual lots.

7.2. Foundation Design

The proposed structures can be supported on conventional or post-tensioned slab/foundation systems. The design of foundation systems should be based on as-graded conditions as determined after grading completion. The following values may be used in preliminary foundation design:

Allowable Bearing: 2000 psf.

Lateral Bearing: 250 psf per foot of depth to a maximum of 2000 psf (level condition). Reduced values may apply for descending slope conditions.

Sliding Coefficient: 0.35

The above values may be increased as allowed by Code to resist transient loads such as wind or seismic. Building code and structural design considerations may govern. Depth and reinforcement requirements and should be evaluated by a qualified engineer.

7.3. Settlement

In addition to the potential effects of expansive soils, the proposed structures should be designed in anticipation of total and differential settlements. The following lot categories are presented based upon anticipated settlement, fill thickness and expansion potential.

Category I

“Very Low to Low” expansion potential and fill depths less than 50 feet. Minimum fill depth meets h/3 criteria where h is the maximum fill thickness.

Total = 3/4 inch
Differential = 3/8 inch in 20 feet

Category II

"Medium" expansion potential and/or fill depths less than 50 feet. Minimum fill depth meets h/5 criteria where h is the maximum fill thickness.

Total = 3/4 inch
Differential = 1/2 inch in 20 feet

Category III

"High" expansion potential and/or fill depths greater than 50 feet.

Total = 1 inch
Differential = 1/2 inch in 20 feet

7.4. Conventional Foundations

Conventional slab-on-grade foundations can be utilized to support the proposed structures for Lot Categories I and II (very low to medium expansion potential). Conventional foundation systems should be designed in accordance with Section 7.2 and Table 7.4 below.

TABLE 7.4
FOUNDATION DESIGN RECOMMENDATIONS

Lot Category	Category I Very Low to Low Expansion Potential	Category II Medium Expansion Potential
Footing Depth Below Lowest Adjacent Finish Grade	12 inches	18 inches
Footing Width	15 inches	15 inches
Footing Reinforcement	No. 4 rebar, one (1) on top and one (1) on bottom	No. 4 rebar, two (2) on top and two (2) on bottom or No. 5 rebar one (1) on top and one (1) on bottom
Slab Thickness	4 inches (actual)	4 inches (actual)
Slab Subgrade Moisture	Minimum of 110% optimum moisture prior to placing concrete.	Minimum of 120% of optimum moisture 24 hours prior to placing concrete.

Footing Embedment Next to Swales and Slopes

If exterior footings adjacent to drainage swales are to exist within five (5) feet horizontally of the swale, the footing should be embedded sufficiently to assure embedment below the swale bottom is maintained. Footings adjacent to slopes should be embedded such that a least seven (7) feet are provided horizontally from edge of the footing to the face of the slope.

7.5. Post-Tensioned Foundations

Post-tensioned foundations may be designed using the values provided in Table 7.5. Design and construction of the post-tensioned foundations should be undertaken by firms experienced in the field. It is the responsibility of the foundation design engineer to select the design methodology and properly design the foundation system for the onsite soils conditions. The slab designer should provide deflection potential to the project architect/structural engineer for incorporation into the design of the structure.

<table>
<tr><th colspan="8">TABLE 7.5
POST-TENSIONED FOUNDATION DESIGN PARAMETERS</th></tr>
<tr><th rowspan="2">Soil Category</th><th rowspan="2">Expansion Index</th><th rowspan="2">Pad No.</th><th rowspan="2">Edge Beam Embedment (inches)*</th><th colspan="2">Edge Lift**</th><th colspan="2">Center Lift**</th></tr>
<tr><th>Em (ft.)</th><th>Ym (in.)</th><th>Em (ft.)</th><th>Ym (in.)</th></tr>
<tr><td>I</td><td>“Low”</td><td>***</td><td>12</td><td>5.4</td><td>0.54</td><td>9.0</td><td>-0.23</td></tr>
<tr><td>II</td><td>“Medium”</td><td>***</td><td>18</td><td>4.6</td><td>0.90</td><td>9.0</td><td>-0.38</td></tr>
<tr><td>III</td><td>“High”</td><td>***</td><td>24</td><td>3.9</td><td>1.26</td><td>7.5</td><td>-0.51</td></tr>
<tr><td colspan="2"><u>Moisture Barrier</u></td><td colspan="6">An approved moisture and vapor barrier should be placed below all slabs-on-grade within living and moisture sensitive areas as discussed in Section 7.6.</td></tr>
<tr><td colspan="2" rowspan="3"><u>Slab Subgrade Moisture</u></td><td>Soil Category I</td><td colspan="5">Minimum of 110 percent of optimum moisture to a depth of 12 inches prior to placing concrete</td></tr>
<tr><td>Soil Category II</td><td colspan="5">Minimum of 120 percent of optimum moisture to a depth of 12 inches prior to placing concrete</td></tr>
<tr><td>Soil Category III</td><td colspan="5">Minimum of 130 percent of optimum moisture to a depth of 12 inches prior to placing concrete</td></tr>
<tr><td colspan="2"><u>Footing Embedment</u>**</td><td colspan="6">Depth of embedment should be measured below lowest adjacent finish grade.
<u>Footings Adjacent to Swales and Slopes:</u> If exterior footings adjacent to drainage swales are to exist within 5 feet horizontally of the swale, the footing should be embedded sufficiently to assure embedment below the swale bottom is maintained. Footings adjacent to slopes should be embedded such that at least 5 feet is provided horizontally from edge of the footing to the face of the slope.</td></tr>
<tr><td colspan="8">NOTES: ** The values of predicted lift are based on the procedures outlined in the Design of Post-Tensioned Slabs-on-Ground, Third Edition and related addendums. No corrections for vertical barriers at the edge of the slab or other corrections (e.g. horizontal barriers, tree roots, adjacent planters) are assumed. <u>The values assume Post-Equilibrium conditions exist (as defined by the Post Tensioning Institute), and these conditions created during construction should be maintained throughout the life of the structure.</u>
*** Final design parameters should be provided in a final grading report and should be based on as-graded soil conditions.</td></tr>
</table>

Post-tensioned slabs should incorporate a perimeter-thickened edge to reduce the potential for moisture infiltration, seasonal moisture fluctuation and associated differential movement around the slab perimeter. The minimum recommended depth of the thickened edge is 12-inches for “low” expansion, 18-inches for “medium” expansion and 24-inches for “high” expansion if existent.

The project foundation design engineer should use the Post-Tensioning Institute (PTI) foundation design procedures as described in 2019 CBC, based upon appropriate soil design parameters relating to edge moisture variation and differential swell provided by the geotechnical consultant at the completion of rough grading operations. For preliminary design and budgeting purposes, Category II design parameters may be assumed. Upon completion of rough grading, finish grade samples should be collected and tested to develop final foundation design recommendations for individual lots.

7.6. Moisture and Vapor Barrier

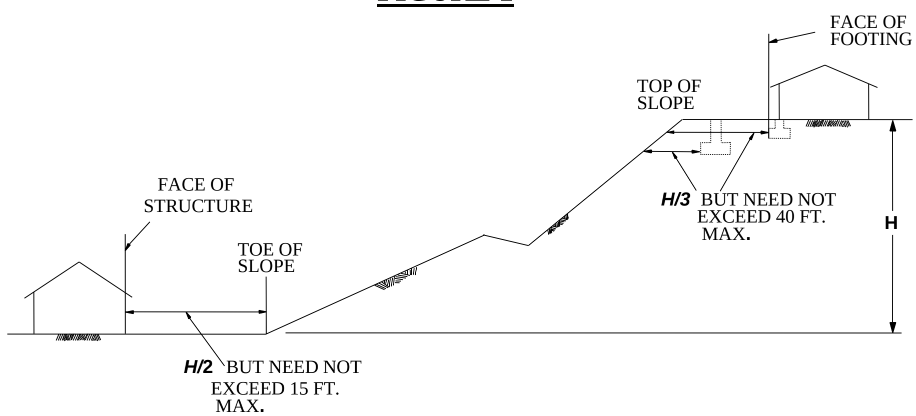
A moisture and vapor retarding system should be placed below the slabs-on-grade in portions of the structure considered to be moisture sensitive. The concrete slab underlayment should consist of a 15-mil vapor retarder, Stego-wrap or equivalent, with all laps sealed per 2019 CBC requirements and the manufacturer's recommendation. The vapor retarder should comply with the ASTM E 1745 - Class A criteria and be installed in accordance with ACI 302.1R-04 and ASTM E 1643 on four inches of clean, angular, open-graded ⅜-inch gravel. The use of this system or other systems, materials, or techniques can be considered, at the discretion of the post-tensioned slab designer, provided the system reduces the vapor transmission rates to acceptable levels.

7.7. Deepened Footings and Structural Setbacks

It is generally recognized that improvements constructed in proximity to natural slopes or properly-constructed, manufactured slopes can, over a period of time, be affected by natural processes including gravity forces, weathering of surficial soils, and long-term (secondary) settlement. Most building codes, including the 2019 CBC, require that structures be set back or footings deepened, where subject to the influence of these natural processes.

For the subject site, where foundations for residential structures are to exist in proximity to slopes, the footings should be embedded to satisfy the requirements presented in Figure 1.

FIGURE 1



7.8. Miscellaneous Foundation Design Recommendations

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

7.9. Earth Pressures for Design of Buried Structures

The recommended active, passive and at rest earth Rankine earth pressures, which may be utilized for design of buried structures with level backfill are as follows:

Static Case

Compacted Artificial Fill, (afc_{90}): phi = 32°, unit wt. = 125 pcf

Level Backfill	Rankine Coefficients	Equivalent Fluid Pressure (psf/lin.ft.)
Coefficient of Active Pressure:	$K_a = 0.31$	38
Coefficient of Passive Pressure:	$K_p = 3.25$	407
Coefficient of At Rest Pressure:	$K_o = 0.47$	59

2 : 1 Backfill	Rankine Coefficients	Equivalent Fluid Pressure (psf/lin.ft.)
Coefficient of Active Pressure:	$K_a = 0.47$	59
Coefficient of At Rest Pressure:	$K_o = 0.85$	106

For rigid restrained walls it is recommended that "At-Rest" values be used. For cantilever retaining walls which can undergo minor rotations active pressures can be used.

The above values may be increased by 1/3 as allowed by Code to resist transient loads. Building Code and structural design considerations may govern.

Seismic Case

In addition to the above static pressures, unrestrained retaining walls supporting more than 6 feet of fill should be designed to resist seismic loading as required by 2019 CBC. The seismic load can be modeled as a thrust load applied at a point 0.4H above the base of the wall, where H is equal to the height of the wall. This seismic load (in pounds per lineal foot of wall) is represented by the following equation:

$$Pe = \tfrac{3}{8} * \gamma * H^2 * k_h$$

Where:

Pe = Seismic thrust load

H = Height of the wall (feet)

γ = soil density = 125 pounds per cubic foot (pcf)

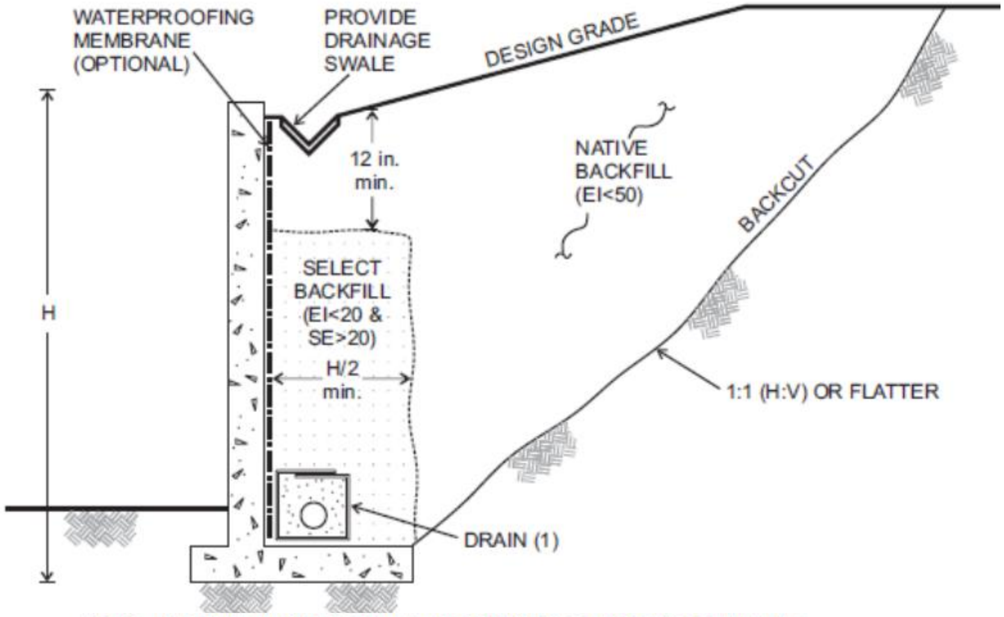
k_h = seismic pseudostatic coefficient = 0.5 * PGA_M (See Table 5.0)

Walls should be designed to resist the combined effects of static pressures and the above seismic thrust load.

Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic pressures. To relieve the potential for hydrostatic pressure wall backfill should consist of a free draining backfill (sand equivalent "SE" >20) and a heel drain should be constructed (see Figure 2). The heel drain should be placed at the heel of the wall and should consist of a 4-inch diameter perforated pipe (SDR35 or SCHD 40) surrounded by 1 cubic feet of crushed rock (3/4-inch) per lineal foot, wrapped in filter fabric (Mirafi® 140N or equivalent).

FIGURE 2

Retaining Wall Backfill and Drainage



NOTES: (1) DRAIN: 4-INCH PERFORATED ABS OR PVC PIPE OR APPROVED EQUIVALENT SUBSTITUTE PLACED PERFORATIONS DOWN AND SURROUNDED BY A MINIMUM OF 1 CUBIC FEET OF 3/4 INCH ROCK OR APPROVED EQUIVALENT SUBSTITUTE AND WRAPPED IN MIRAFI 140 FILTER FABRIC OR APPROVED EQUIVALENT SUBSTITUTE

Proper drainage devices should be installed along the top of the wall backfill, which should be properly sloped to prevent surface water ponding adjacent to the wall. In addition to the wall drainage system, for building perimeter walls extending below the finished grade, the wall should be waterproofed and/or damp-proofed to effectively seal the wall from moisture infiltration through the wall section to the interior wall face.

The wall should be backfilled with granular soils placed in loose lifts no greater than 8-inches thick, at or near optimum moisture content, and mechanically compacted to a minimum 90 percent of the maximum dry density as determined by ASTM D1557. Flooding or jetting of backfill materials generally do not result in the required degree and uniformity of compaction and, therefore, is not recommended. No backfill should be placed against concrete until minimum design strengths are achieved as verified by compression tests of cylinders. The geotechnical consultant should observe the retaining wall footings, back drain installation, and be present during placement of the wall backfill to confirm that the walls are properly backfilled and compacted.

7.10. Pool Design Recommendations

The following preliminary recommendations are provided for consideration during design and construction of swimming pools and pool decks. The equivalent fluid pressure to be used for pool design should be based on the values provided in Section 7.9 for compacted fill.

Concrete deck slabs should be a minimum of 4 inches thick. Consideration should be given to adding reinforcement in order to mitigate the potential for cracking and possible water intrusion into the underlying soil. The deck slab should be separated from the pool and the joint should be properly sealed to prevent any moisture from reaching the subgrade. Expansion and contraction joints should be properly sealed to prevent moisture intrusion and should be properly maintained. Consideration should be given to constructing a deepened perimeter edge footing at the bond beam of the pool. The edge should be a minimum of 8 inches wide by 18 inches deep and be reinforced at the top and bottom with a minimum of one No. 4 rebar. A minimum deepened edge of 18 inches should also be constructed adjacent to any landscaped areas.

The subgrade should be moisture conditioned to a minimum of 120% of the optimum moisture content to a depth of 12 inches prior to placement of concrete.

Plumbing trenches and drain lines should be properly backfilled to prevent water from traveling along the trench backfill. A cutoff wall of compacted impermeable materials or slurry should be installed at the edges of where trenches extend below the pool deck.

7.11. Corrosion

Testing should be conducted on the near-surface soils after grading completion and recommendation should be provided at that time.

Preliminary test results presented in the referenced reports and those reviewed during current mass grading operations indicated that the onsite soils have low concentrations of soluble sulfate, corresponding to class S0 exposure when classified in accordance with ACI 318. Sulfate resistant concrete is not anticipated to be required.

Resistivity tests performed during mass grading indicated that onsite soils are considered corrosive to buried metallic materials. In the past on similar projects, corrosion protection typically consisted of non-metallic piping for water lines to and below the slabs or by installing above slab plumbing. Consultation with a corrosion engineer should be considered for the design of future site improvements if additional recommendations are needed.

Advanced Geotechnical Solutions, Inc. appreciates the opportunity to provide you with geotechnical consulting services and professional opinions. If you have any questions, please contact the undersigned at (619) 867-0487.

Respectfully Submitted,
Advanced Geotechnical Solutions, Inc.

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CEG 2536, Reg. Exp. 5-31-23

Distribution: (1) Addressee (pdf)

Attachments: Appendix A - References
Plate 1 - Preliminary As-Graded Geologic Map

APPENDIX A
REFERENCES

APPENDIX A
REFERENCES

American Society of Civil Engineers, 2016, ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures.

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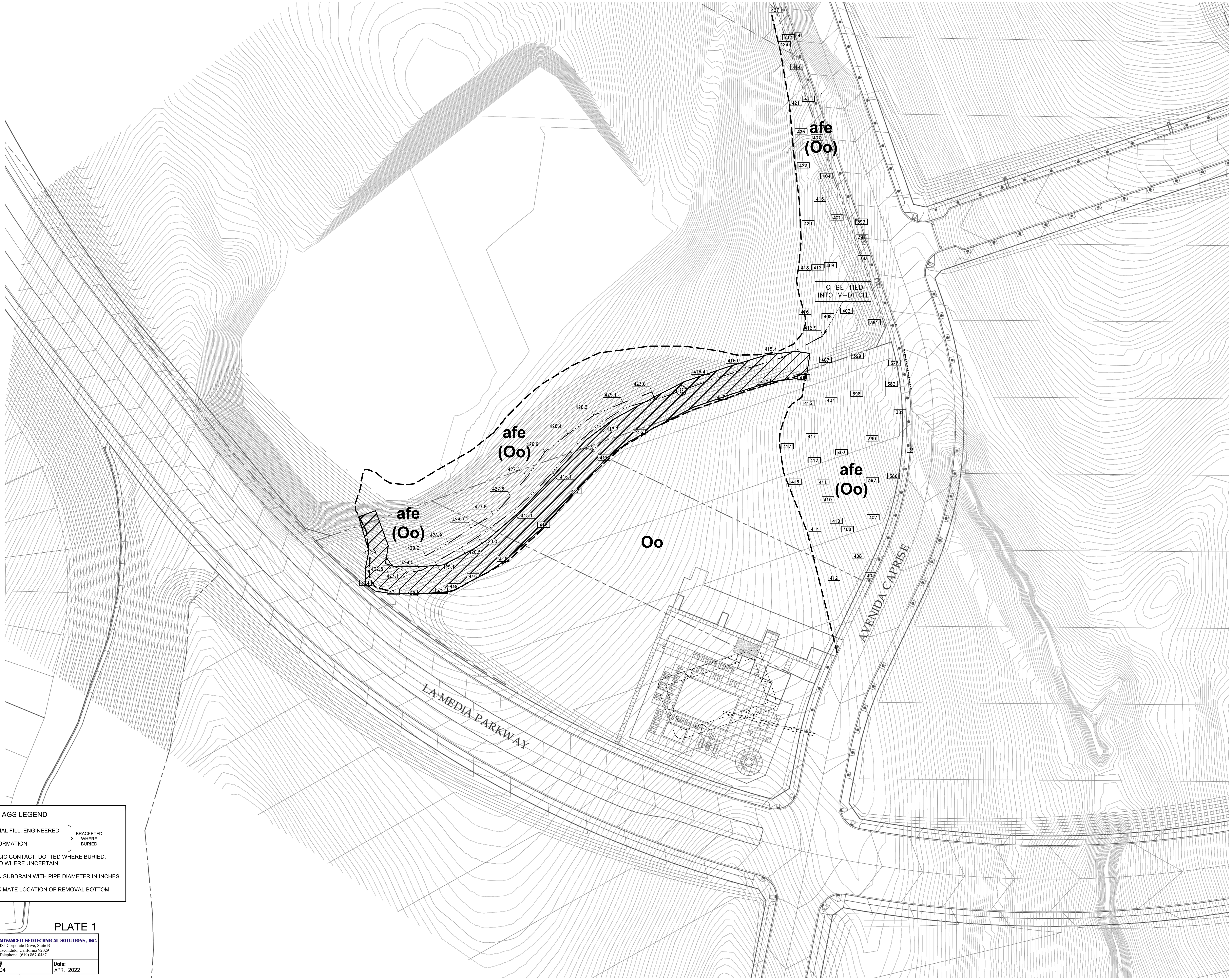

AGS LEGEND

afe	ARTIFICIAL FILL, ENGINEERED	BRACKETED WHERE BURIED
Oo	OTAY FORMATION	
— — · · ·	GEOLOGIC CONTACT; DOTTED WHERE BURIED, QUERIED WHERE UNCERTAIN	
⊢ · · · ⑥ —	CANYON SUBDRAIN WITH PIPE DIAMETER IN INCHES	
403	APPROXIMATE LOCATION OF REMOVAL BOTTOM	

PLATE 1

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Project#	Report#	Date:
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BrightView Design Group

Date: July 7, 2017

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