PRELIMINARY GEOTECHNICAL INVESTIGATION,
THE MASTER'S COLLEGE,
SANTA CLARITA, CALIFORNIA

Prepared For:

The Master's College
21726 Placerita Canyon Road
Santa Clarita, California 91321

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October 5, 2005

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Work Order: 2668-0-0-10
Log Number: 23978

Subject: **Preliminary Geotechnical Investigation, The Master's College, Santa Clarita, California**

INTRODUCTION

The following geotechnical report contains the results of our geotechnical evaluation for the proposed development at Master's College in Santa Clarita, California. The geotechnical evaluation includes a hazards evaluation and the presentation of foundation recommendations.

PROPOSED DEVELOPMENT

Discussions with Michael Browers of Behr Browers Architects, along with the preliminary grading plan for the site and the MacArthur Chapel dated February 22, 2005 prepared by Gannfors & Associates, Inc. were used for preparation of this report. Dockweiler Drive will be extended in a westerly direction necessitating cut and fill slopes to support the road. Maximum fill slope heights aligned at 2(h):1(v) gradients will be on the order of 160 feet with a maximum fill depth of 90 feet. Proposed cut slopes will also be aligned at 2(h):1(v) gradients with a maximum height of 95 feet and a maximum cut depth of 80 feet. Southerly of the extension of Dockweiler Drive, 35 residential pads will be constructed along with the extension of a parallel road (Deputy Jake Drive). Proposed construction on the Master's College property includes MacArthur Chapel, future classroom and administration buildings, and dormitories. Appurtenant parking, driveways and landscaping are also proposed. We anticipate that sewage disposal will be via connection to an existing sanitary sewer system.

GEOTECHNICAL CONSIDERATIONS

The project site is located atop and at the base of a westerly trending ridgeline with several intervening valleys. The state seismic hazards map has identified areas of potential seismically induced landslides in the hillside areas and potential liquefaction in the alluvial low lying areas within the limits of the project.

Geotechnical considerations include slope stability evaluation for fill, cut and natural slopes; settlement evaluation of deep fills; and liquefaction evaluation of low-lying alluvial areas.

Existing artificial fill deposits are located in the southeastern portion of the site in association with the development of the adjacent residential Tract 53114 (Geolabs, 2003). A fill slope on the order of 100 feet in height aligned at a 2(h):1(v) with a maximum fill depth of approximately 50 feet descends offsite from Lots 1-5, Tract 53114 to the incised drainage. The proposed development in this area indicates that additional artificial fill will be placed on this fill slope to support Lots 24 through 26.

SCOPE OF WORK

Gorian and Associates, Inc. strived to perform the geotechnical evaluation in general accordance with the generally accepted geotechnical engineering practice for the area. The scope of services described below was performed under the direction of a State Certified Engineering Geologist and Registered Geotechnical Engineer. Our evaluation was performed to characterize the overall geologic/geotechnical engineering setting via a program of archival research, geologic field mapping, subsurface exploration, laboratory testing and geotechnical analyses. General geotechnical site parameters and preliminary geotechnical recommendations for site preparation, grading and foundation designs are presented. Our scope of services included:

Archival Research

Pertinent geotechnical data in our files was reviewed including regional geologic maps, geotechnical / geologic hazard maps, and Alquist-Priolo earthquake fault-rupture hazard zone maps. Readily available geologic and geotechnical reports for the site and vicinity were reviewed and pertinent geotechnical data utilized in our evaluation.

Geologic Reconnaissance

A geologist from our office performed geologic field mapping of the proposed development area to substantiate and supplement the geologic data shown on available literature. Emphasis of the field mapping was focused on identifying geologic hazards and structure that may impact development of the property.

Geologic Analyses

The results of the archival research and geologic mapping are presented on the attached Geotechnical Map (Plate 1) and depict the approximate distribution of earth units and pertinent cross sections. Cross sections were constructed to illustrate the proposed development and building areas in relation to the geologic hazards and geologic structure.

Field Exploration

The field exploration consisted of drilling three (3) hollow-stem auger borings (HS-1, HS-2 and HS-3) to depths ranging from 26½ feet (HS-1) to 42½ feet (HS-3) below the existing ground surface utilizing a subcontractor supplied and operated truck mounted CME 75 drill rig equipped with 8-inch diameter augers. Drive samples were obtained utilizing a 140 pound automatic hammer dropped 30 inches. Standard Penetration Test (SPT) samples were obtained at a maximum vertical spacing of 5 feet for the liquefaction evaluation. All three borings were terminated in underlying bedrock materials.

In addition to the hollow-stem auger borings, ten (10) bucket auger borings were excavated to depths ranging from 41 feet (B-10) to 100 feet (B-7) below the existing ground surface utilizing a subcontractor supplied and operated truck mounted Wilkinson RH-42 drill rig equipped with a 24-inch diameter bucket auger. Geologists from this office logged the underlying materials and obtained selected soil and bedrock (bulk and relatively undisturbed) samples for laboratory analyses. A geologist from this office entered each of the bucket auger borings for detailed down-hole logging of soil/bedrock contacts and bedrock structure.

Eight (8) backhoe trenches were excavated at selected locations on the site to evaluate removals and the landslide postulated by Earth Systems in 2005. These exploratory excavations ranged in depth from 4 feet (T-6) to 11 feet (T-3) below the existing ground surface. The trenches were excavated by a subcontractor supplied and operated rubber tire backhoe with a 24 inch bucket. A geologist from this office logged the underlying soil and bedrock materials. Trench T-1 was excavated in the graben of the postulated landslide and T-2 was excavated near the toe. No evidence of a graben or scarp was encountered and the observed in-place bedrock consisted of dense sandstone. Trenches T-3 through T-8 were excavated in the incised drainages to evaluate alluvial removals. The anticipated depths of unsuitable soils to be removed in these drainages are shown on the attached Geotechnical Map (Plate 1).

At the end of excavation, depth to groundwater, if any, was noted and the borings and backhoe trenches were backfilled with the excavated materials. The backfill may settle with time and therefore a representative of the Master's College should observe the areas of exploration periodically and any depressions that develop should be backfilled.

Laboratory Testing

Laboratory testing was performed on selected soil and bedrock samples obtained during the field exploration. Testing was performed to obtain in-situ moisture content and density, shear strength parameters, consolidation characteristics, expansion potential, maximum dry density/optimum moisture content relationships, grain size distribution, R-values and corrosivity potential. The corrosivity and R-value testing were performed by subcontracted engineers.

Engineering Analyses and Report Preparation

The results of the field and laboratory programs were used in engineering analyses to develop geotechnical recommendations for site development and construction. The results of our findings are provided in this report that includes:

1. A Geotechnical Map (Plate 1) showing the site, the approximate distribution of earth units, and the approximate locations of the exploratory borings and backhoe trenches and Geotechnical Cross Sections 1-1' through 8-8' (Plate 2).
2. A description of soil and groundwater conditions, as encountered during the subsurface exploration, including Logs of Subsurface Data (Appendix A).
3. A description of the laboratory testing program, including test results (Appendix B).
4. Discussion and preliminary geotechnical recommendations regarding:
    a) Seismic analyses for the site and seismic design criteria;
    b) Evaluation of the secondary aspects of ground shaking (liquefaction, lateral spreading, seismic settlement, etc.);
    c) Total and differential settlement from static loading.
    d) Slope stability.
    e) Site preparation and grading recommendations.
    f) Preliminary foundation design and construction recommendations.
    g) Preliminary pavement design criteria.

CHAPEL RECOMMENDATIONS

Foundation and seismic design recommendations specifically for the MacArthur Chapel have been provided under separate cover.

SITE DESCRIPTION

LOCATION AND PHYSIOGRAPHY

The Master's College is located at 21726 Placerita Canyon Road in the Santa Clarita area of the County of Los Angeles, California. The approximate site location is shown on the attached Vicinity Map (Figure 1). The irregularly shaped hillside property is situated between Placerita Canyon Road to the north and Newhall Creek to the south. Existing residential homes are located just off site to the east-southeast. Dockweiler Drive extends from the existing residential homes towards the west where it terminates on the top of the east-west trending ridge.

The area of the proposed residential development is located on the generally southeast-northwest trending bedrock hillside areas and at the toe of the hillside on both the northern and southern sides. The hillside areas are moderately steep rolling bedrock ridges and ridge spurs with slope gradients ranging from approximately 2(h):1(v) on the sides of the hillside to nearly level on the top of the ridges. The hillside areas are covered with a moderate to dense growth of seasonal weeds and grasses with scattered patches of native shrubbery.

The southern side of the hillside adjacent to Newhall Creek has several alluvial filled tributary canyons that drain westward towards the adjacent creek. One of these southerly draining incised ravines has been filled with artificial fill in association with the underlying Metropolitan Water District (MWD) waterline (Foothill Feeder/Placerita Tunnel). The MWD waterline is approximately 20 feet in diameter and traverses the property in a northwest-southeast trend on the southern side of the hillside. Based on our understanding, the waterline was tunneled under the western portion of the site (see Plate 1). The cover over this tunnel appears to vary from approximately 125 feet in the hillside areas to 50 feet in the area of the filled drainage that is currently used as a playfield by the college.

Vegetation observed within the southern portion of the property includes coastal sage scrub assemblages, yucca, seasonal grasses and oak trees.

The northern side of the hillside has been developed for the college including dorms, classrooms, administration buildings, and surface parking and drive areas. Incised drainages on the northern side of the hillside are filled with mature oak trees and these drainages flow towards the north to existing storm drain structures. Total relief in the area of the proposed development is on the order of 260 feet.

BACKGROUND

Based on an archival review, the property was recently evaluated by Earth Systems (Earth Systems, 2005). At that time, the geotechnical consultant characterized the hillside area of the site as being underlain entirely by bedrock of the Saugus Formation mantled with slope wash. The lower reaches of the property including the incised drainages were characterized as being filled with alluvium and the area adjacent to Newhall Creek was characterized as being filled with stream channel deposits. Localized deposits of artificial fill were also delineated. Structural data presented on their Geologic Site Plan indicates that the bedrock is inclined in a northerly direction at low to moderate angles (5° to 13°). A small landslide was delineated on the southern side of the hillside area in the central portion of the site along the proposed alignment of the extension of Dockweiler Drive. This landslide was indicated as moving in a southerly direction.

Based on geologic data and aerial photographs, Earth Systems found no evidence of faulting and concluded that the hazard posed by fault rupture is low. Based on a review of FEMA maps, Earth

Systems determined that the southern side of the site adjacent to Newhall Creek as well as the northern side of the site adjacent to Placerita Creek is situated within a 100-year flood zone; however the hazard posed by flooding is confined to a relatively small area adjacent to the aforementioned creeks. They concluded that the site could be developed as proposed with the most significant geologic hazard being seismic shaking which can be mitigated utilizing standard design codes.

REGIONAL GEOLOGIC SETTING

The subject site is situated within the Transverse Range Geomorphic Province, in the eastern portion of the Ventura Basin which is separated from the Soledad Basin and the Ridge Basin by the active San Gabriel fault zone. This fault zone traverses the area in a generally northwest-southeast trend approximately 9,000 feet to the north of the site. The Ventura Basin was subsequently filled with sediments eroding from the adjacent uplifted bedrock blocks which form the surrounding mountains. These sediments, which consist primarily of sand and gravel, contain abundant granitic and volcanic rocks suggesting a source in the San Gabriel and Santa Susana Mountains. The Saugus Formation is a continental deposit overlying marine Pico Formation of Pliocene age and overlain by upper Pleistocene deposits of the ancestral Santa Clara River (South Coast Geological Society, 1982) including the Pacoima Formation.

SITE GEOLOGY

Based on a review of pertinent geologic data, field mapping, and subsurface exploration, the subject site is underlain by bedrock of the Quaternary-age Pacoima Formation (Qp) mantling bedrock of the Plio-Pleistocene-age Saugus Formation (TQs) (see Regional Geologic Maps, Figures 2 (after Treiman) and 2A (after Dibblee)). The Saugus Formation extended to the maximum depth explored, 100 feet below the existing ground surface

Soil units on the property include relatively thin mantles of Quaternary-age alluvium (Qal), Quaternary-age stream channel deposits (Qsc), artificial fill (af), and both topsoil and colluvium. Due to the relatively thin nature of the topsoil and colluvial deposits, they are not shown on the attached Geotechnical Map (Plate 1). Descriptions of these units are presented below and in the attached Logs of Subsurface Data (Appendix A).

Based on a review of the State of California landslide hazard maps (CDMG, 1987), all of the proposed development area is situated in an area marginally susceptible to debris flow. In other words evidence of debris-flow activity is uncommon but may occur locally.

SAUGUS FORMATION (TQs)

Bedrock of the Plio-Pleistocene-age Saugus Formation underlies the Pacoima Formation to the maximum depth explored. The Saugus Formation is commonly exposed on the southern side of the subject hillside area and in the northeastern portion of the site where not covered with the draping Pacoima Formation. The Saugus Formation represents paleo-Santa Clara River floodplain deposits and adjacent interfingering tributary and alluvial fan sediments (CDMG, 1987).

The Saugus Formation, as encountered during our subsurface exploration, generally consists of yellowish brown silty to clayey fine- to coarse-grained sandstone with some gravel and cobbles in a damp to moist condition. Typically, these sandstone units are friable. Granitics, metavolcanics, quartzite and Pelona schist clasts were noted. Locally, the sandstone is interbedded with yellowish brown clayey siltstone to sandy siltstone and reddish brown to yellowish brown mudstone (mudstone consists of particles 1/3 sand, 1/3 silt and 1/3 clay) to sandy claystone in a damp to moist condition.

Structurally, based on field mapping, subsurface exploration and a review of pertinent geologic data, the Saugus Formation is inclined to the north-northwest at moderate angles (12° to 16° after CDMG, 1987) to low angles (2° to 10° after Dibblee, 1996). The Saugus Formation, as observed in outcrops and during downhole logging operations, is crudely bedded to locally well defined, but typically exhibits gradational contacts. Some of the beds can be correlated across the site while others appear to be interfingering and grade laterally. The top of the Saugus Formation /base of the Pacoima Formation is commonly a distinct erosional surface as observed in borings B-1, B-2, B-4, B-6, B-7, B-8, and B-9. The top of the Saugus Formation appears to be a poorly developed paleosol consisting of reddish brown grading to yellowish brown mudstone with manganese oxide staining to sandy claystone to locally yellowish brown clayey to silty fine to coarse-grained sandstone to cemented sandy to clayey siltstone.

Cross section 1-1' illustrates the relationship between the ridge capping Pacoima Formation and the underlying Saugus Formation. This cross section illustrates the apparent dip of the bedding of the Saugus Formation inclined in a northwesterly direction at low angles (1° to 7°) while the true dip is generally inclined in a northerly direction at low angles (2° to 10°). Some fluctuations in bedding including cross bedding were noted and appear to be the result of a fluctuating depositional environment, not tectonic activity.

PACOIMA FORMATION (Qp)

Bedrock of the Quaternary-age Pacoima Formation mantles the majority of the upper reaches of the generally northwest-southeast trending ridge and drapes over the hillside on the northern side to the toe of the hillside. This formation has been designated as "Older Dissected Surficial Sediments or Qog" after Dibblee, 1996; however these "high terrace deposits of sand and gravel" have been assigned to the Pacoima Formation (of Oakeshott, 1958) by Treiman (CDMG, 1987). For this report, the nomenclature used by Treiman will be used.

The Pacoima Formation, as encountered in our exploratory bucket auger borings, was found to vary in thickness from 5 feet (B-6) to more than 70½ feet (B-5). These bedrock materials were encountered in all of the bucket auger borings with the exception of B-10. The bedrock generally consists of light yellowish brown to yellowish brown to brown silty to clayey fine- to coarse-grained sandstone with some gravel, cobbles and rare boulders in a damp to moist and commonly friable condition. Locally the sandstone is interbedded with yellowish brown clayey siltstone to sandy claystone in a moist to very moist condition. Clasts incorporated in this formation generally consist of subangular to well rounded metavolcanics, granitics, and quartzite. The sandstone units are locally crossbedded and locally have magnetite laminations. Bedding is generally crude to locally well defined.

The Pacoima Formation represents a period of basin filling with sediments derived from the erosion of the Santa Susana Mountains and northern sources, interfingering with sediments derived from the paleo-Santa Clara River (CDMG, 1987).

Structurally, the Pacoima Formation rests unconformably on the underlying Saugus Formation, however, locally appears to be conformable where both bedrock units are nearly horizontal. Based on a review of the CDMG map, the Pacoima Formation is horizontally bedded. Geologic data obtained from the downhole logging operations indicate that the bedrock is generally inclined in a northerly direction at low angles (0° to 10°), although some variation and fluctuation in bedding was noted with bedding inclined in a southerly and easterly direction also at low angles (3° to 7°). The base of this formation is usually a sharp yet irregular erosional contact that is nearly horizontal on the top of the ridges and inclined in a northerly direction at low to moderate angles (6° to 14°) where the bedrock drapes over the hillside to the north.

ALLUVIUM (Qal)

Quaternary-age alluvium typically fills the bottom of the valley floors and portions of the incised drainages. These water transported materials are easily visible in the Newhall Creek area and locally on the site where erosion has exposed these deposits. Alluvium was encountered in all three of the hollow stem auger borings (HS-1 through HS-3) and was found to vary in thickness from 17½ feet (HS-1) to 34 feet (HS-3). The alluvium was further evaluated by excavating six (6) backhoe trenches in the incised drainages on the southern side of the hillside and was found to vary in thickness from 1½ feet (T-8) to 8½ feet (T-3).

The alluvium on the northern side (HS-1) of the hillside area generally consists of dark gray silty fine sand in a damp and loose condition grading downward to yellowish brown clayey fine to coarse sand with some gravel and cobbles in a damp and dense condition. The lower portion of the alluvium generally consists of light yellowish brown silty fine to coarse sand with gravel in a slightly moist and dense condition.

The alluvium on the southern side (HS-2, HS-3) of the site adjacent to Newhall Creek generally consists of very dark grayish brown to brown grading to yellowish brown fine sand to silty sand to gravelly sand in a damp to moist and loose to very dense condition locally interstratified with dark reddish brown to brown sandy to clayey silt in a damp and hard condition.

The backhoe trenches were excavated just uphill from the hollow stem borings and in drainages inaccessible to a drill rig. These trenches encountered alluvium generally consisting of brown silty fine to coarse sand in a porous, damp and loose condition overlying brown to yellowish brown clayey fine to coarse sand with gravel to boulders in a damp and medium dense condition yet porous with pores to 1/8". Typically, the base of the alluvium is defined with a cobble to boulder lag deposit and a sharp erosional contact with the underlying bedrock.

STREAM CHANNEL DEPOSITS (Qsc)

Soil deposits in the active Newhall Creek are referred to as stream channel deposits. These deposits were not investigated, as they are not within any area of the proposed construction. However, based on visual examination, these deposits generally consist of sand and gravel in a damp and loose condition. These materials are similar to the alluvium; however, the stream channel deposits have been reworked during periods of heavy runoff.

COLLUVIUM

Soils that have been deposited under the influence of gravity are referred to as colluvium. Colluvium is commonly deposited down slope faces and in incised ravines. Colluvium was encountered in boring B-10, which was located in an incised ravine that was subsequently covered with artificial fill. The colluvium was found to be on the order of 3½ feet in thickness in B-10 and consisted of very dark grayish brown clayey sand with some gravel and cobbles in a moist and medium dense condition. The contact with the underlying bedrock is gradational over 12 inches.

TOPSOIL

Topsoil generated from the in-place weathering of the bedrock materials typically mantles the bedrock materials. The topsoil encountered in boring B-5 and trenches T-1 and T-2 is on the order of 1 to 1½ feet in thickness and generally consists of brown silty fine to coarse sand with some gravel and cobbles in a damp and medium dense condition. Seasonal weeds and grasses typically extend their root systems through the topsoil materials.

ARTIFICIAL FILL (af)

Artificial fill associated with construction of the existing college is generally confined to the northern side of the hillside in close proximity to the existing buildings and access roadways. These fill deposits were not a part of the scope of work and therefore were not investigated. The approximate limits of the artificial fill are shown on the attached Geotechnical Map (Plate 1).

Artificial fill was encountered in our hollow stem boring HS-1, which was excavated in an area currently used to dispose of soils and construction debris within the college property. The artificial fill here was found to be 2½ feet in thickness and generally consists of dark gray silty fine to coarse sand with some concrete and asphaltic concrete debris in a damp and loose condition.

Additional artificial fill deposits were encountered in boring B-10. This boring was located in the northeastern corner of the playfield on the southern side of the hillside. As previously noted, this playfield was manufactured as a result of "permanent required fill to an elevation of 1325" during the construction of the Placerita Tunnel based on a review of plans and profiles provided to us by the MWD. The current elevation of the playfield ranges from elevation 1325 to 1330, so it is possible that some minor artificial fill has been placed over the fills made by MWD. The artificial fill encountered in B-10 was found to be on the order of 5 feet in thickness and generally consists of brown clayey to silty fine to coarse sand with some gravel and cobbles in a damp and medium dense condition. Maximum depth of fill within the playfield area based on MWD plans is approximately 35 feet.

Additional artificial fill deposits were encountered during the backhoe trenching operations. Artificial fill was encountered in Trench T-8 and was found to be on the order of 1 foot in thickness and generally consists of brown silty fine sand with concrete metal, and plastic debris in a damp and loose condition. The approximate limits of this deposit are shown on the attached Geotechnical Map (Plate 1).

Additional artificial fill deposits (previously engineered fill) are located in the southeastern portion of the site in association with the development of the adjacent residential Tract 53114 (Geolabs, 2003). A fill slope on the order of 100 feet in height aligned at a 2(h):1(v) with a maximum depth of approximately 50 feet descends offsite from Lots 1-5, Tract 53114 to the valley floor.

Additional areas of artificial fill may exist on the site but were not investigated or mapped as they are concealed.

GROUNDWATER

Groundwater was encountered as seepage in borings HS-1 at 17 feet, B-2 at 20 to 22 feet, B-6 at 22 feet, B-8 at 52 feet, B-9 at 23 to 24 and at 52 feet, and B-10 at 23 feet below the existing ground surface. Typically, the seepage was confined to silty fine to coarse-grained sandstone units that are underlain by fine-grained materials such as claystone (mudstone) and siltstone. No other groundwater was observed on the site. No surface water was observed flowing in the adjacent Newhall Creek during our field exploration program.

LANDSLIDES

No landslides are present within or near the site nor are any shown on regional geologic maps. A suspected landslide previously delineated (Earth Systems, 2005) was investigated during our field exploration program and it was concluded that this postulated landslide does not exist. Trenches T-1 and T-2 were excavated at the top and toe of the postulated landslide and a thin soil cover over dense in-place sandstone was encountered. No evidence of landsliding was encountered (ie: thick soil development, scarp, graben soils, sheared clay).

FAULTING AND SEISMICITY

Faults identified as active or potentially active in published geologic literature (Hart, et. al, 1999) are not known to be present within or adjacent to the subject site. Our recent field exploration revealed no indication of active or potentially active faults. However, the project site is situated in the seismically active Transverse Ranges and can be expected to experience strong ground shaking from earthquakes generated on active regional faults, as evidenced by the strong groundshaking generated by the January 17, 1994 Northridge earthquake (magnitude 6.7).

The Northridge earthquake produced strong ground shaking and localized substantial damage within the Santa Clarita-Newhall area. The earthquake occurred on a previously unknown thrust fault (Chang, et. al., 1994) with the epicenter located approximately 14 miles southwest of the subject site. Peak horizontal ground acceleration in the vicinity of the site was estimated to be on the order of 0.45g-0.50g (U.S.G.S., 1994).

A quantitative estimation of the hazard of earthquake ground shaking (strong ground motion) possible at the site was evaluated using the deterministic analysis program EQFAULT (Blake, 2000). Data obtained from the analyses are presented in Appendix C along with a map of earthquakes within 65 miles of the site.

According to the deterministic analyses, the nearest active faults are the San Gabriel fault (2.9 miles), the Holser Fault (3.2 miles), the Santa Susan fault (3.4 miles) and the Sierra Madre fault (4.4 miles). The deterministic analyses were run twice – once with Pleistocene soil and once with Holocene soil. Of the two soil types, the Holocene soil produced the greatest peak acceleration values, which are presented below.

According to the deterministic analyses, the San Gabriel fault is capable of producing a magnitude 7.2 (Mw) earthquake and generating peak ground accelerations of 0.76 g. The Holser Fault is believed to be capable of producing a magnitude 6.5 (Mw) earthquake with peak accelerations of 0.89g. The Santa Susan Fault is capable of a magnitude 6.7 earthquake with peak accelerations of 0.92g, while the Sierra Madre fault is capable of producing a magnitude 6.7 (Mw) earthquake and peak ground accelerations of 0.81g.

Based on the Seismic Hazard Evaluation Report for the Newhall Quadrangle published by the California Division of Mines and Geology (CDMG), the project site has a 10% chance in 50 years (475 year return period) of experiencing accelerations of 0.75g for alluvium conditions and 0.78g for soft rock conditions based on a predominant earthquake of magnitude 6.6 (Mw) at a distance of 2 km from the site (CDMG, 1997),

Secondary earthquake hazards include liquefaction, seismically induced settlement or differential compaction, tsunami, seiches and flooding from dam failures. Due to the elevated topography of the site, flooding, tsunami and seiches are not expected to present a risk to the site. However, the State of California Seismic Hazard Zones map for the Newhall Quadrangle (CGS, 1998) shows that portions of the site may be susceptible to liquefaction and/or earthquake induced landslides. These issues, along with settlement and compaction, have been evaluated and are addressed in subsequent sections of this report.

No direct evidence of active or potentially active faulting was observed within the subject property. The potential for surficial ground rupture due to faulting at the site is considered remote during the lifetime of the project.

SLOPE STABILITY

Manufactured fill and cut slopes will be constructed at a maximum gradient of 2(h):1(v) or flatter within the proposed development area. Maximum heights are on the order of 160 feet and 95 feet for fill and cut slopes, respectively. Stability analyses were performed on the highest fill and highest cut slopes within the proposed development. Surficial stability of manufactured slopes was also evaluated.

The computer program XSTABL (Interactive Software Designs, Inc.) which utilizes Bishop's simplified method of slices for rotational failures and Simplified Janbu Force Equilibrium method with Rankine active and passive pressure angles for translational failures was used to evaluate gross slope stability of the proposed cut and fill slopes discussed above.

The proposed fill slopes should be constructed with keys and benched into competent materials. The highest cut slope, as shown on Section 4-4', will require stabilization due to adverse bedding in the area of the cut. Other cut slopes may require stabilization fills. A detailed description including the results of the gross and surficial stability analyses is presented in Appendix D. A discussion regarding the stabilization is included in the Conclusions and Recommendations section below.

LIQUEFACTION AND SEISMIC SETTLEMENT

Many of the proposed fill slopes will be constructed on the existing alluvial soils. Liquefaction and the potential for seismic settlement were evaluated for the alluvial soils to evaluate the impact of potential settlement on the proposed development. Field and laboratory data from borings HS-1 through HS-3 was used to evaluate the potential for liquefaction and seismic settlement.

Liquefaction can occur when saturated, loose, granular soils are subjected to excessive ground vibrations. During liquefaction, excessive pore pressure increases cause these soils to lose strength. This may result in mobilization of the soil causing large total or differential settlements, lateral spreading, and/or surface manifestations such as loss of bearing capacity, artesian water flow, and sand boils. The analyses conducted for this investigation were completed per CDMG (1997) Special Publication 117.

Due to the lack of groundwater in the alluvial soils, liquefaction and the associated saturated sand settlement are not considered likely. However, seismic settlement of dry sands (unsaturated sands above groundwater) is likely within the upper alluvial soils. The potential for dry sand seismic settlement was evaluated using the procedure proposed by Pradel (1998) which uses a series of equations to determine the volumetric strain induced in a soil layer based on the equivalent corrected N-values obtained during the field investigation and the earthquake parameters. The earthquake parameters used in the analyses (magnitude of 6.6 and acceleration of 0.75g) are based on the CDMG Seismic Hazard Report for the Newhall Quadrangle (1997). After the recommended removals are performed, seismically induced settlement of dry sands should be less than ½ inch. The liquefaction calculations are presented in Appendix E.

HYDROCONSOLIDATION

Hydroconsolidation is the phenomena in which naturally occurring soils collapse or consolidate upon inundation with water. Alluvial soils and bedrock were evaluated for the potential to hydroconsolidate based on the results of load consolidation tests, overburden pressures, and the assumed design groundwater levels. The results of the evaluation indicate that the possibility for hydroconsolidation within the site is relatively low.

IMPACTS ON TUNNEL/MWD RIGHT OF WAY

A portion of the proposed development is located over the Foothill Feeder/Placerita Tunnel. As stated above, the cover over the tunnel appears to vary from approximately 125 feet in the hillside areas to 50 feet in the area of the filled drainage that is currently used as a playfield by the college. Based on a Plan and Profile of the tunnel (Metropolitan Water District of Southern California Specification No. 750, Sheet

No. C-12), in the valley area filled for use as a playfield, a fill to a minimum elevation of 1325 is required. To the northwest and southeast of the valley, the cross section shows existing hillside areas.

Within the 250 foot wide tunnel easement, a cut up to 15 feet in thickness is proposed along the planned Dockweiler Drive alignment. The cut will reduce the elevation of the ground surface to 1340, approximately 15 feet above the minimum elevation. The tunnel should not be impacted by the proposed cut.

There are also 2 areas of fill located within the easement. One, shown in plan by Cross Section 8-8', consists of a 2(h):1(v) fill slope for support of Dockweiler Drive. The fill will be placed over existing fill soils placed over the tunnel and will range from a thin veneer at the toe of the slope to as thick as 20 feet at the top of slope. Because the soil cover over adjacent portions of the tunnel exceed the height proposed, the impacts from the proposed fill in the area of Cross Section 8-8' on the tunnel are expected to be minimal. The second area of proposed fill located in the tunnel easement is shown on Cross Section 7-7' and consists of a 2(h):1(v) slope extending up towards the residential development. This area corresponds to the transition from the tunnel to the Newhall siphon. As shown on the Geotechnical Map and Cross Section 7-7', the proposed fill slope will be supported entirely by Saugus Formation bedrock. Proposed fills entirely supported by bedrock are not expected to negatively impact the existing water line. It should be verified during construction that the slope is supported by bedrock. If portions are supported by fill, additional analyses should be conducted to evaluate the impacts on the waterline.

STATIC SETTLEMENT

The potential for static settlement as a result of placing the deep fills was evaluated. In general, the recommendations contained herein will result in very little settlement of the existing materials. For instance, settlement in bedrock materials will be very small. In areas of fill placement, all artificial fills and colluvial soils will be removed to bedrock, removing the potential for soil settlement during fill placement. Additionally, loose alluvial soils will be removed prior to fill placement. Therefore, the settlement calculations performed were completed to evaluate the potential for static settlement within the proposed fills. Maximum fill thicknesses in the eastern portion of the site to the north of Dockweiler Drive will range from approximately 50 to 85 feet. Fills on the south side of Dockweiler Drive will reach as thick as 70 feet in the canyon areas. To keep settlement within tolerable limits, fill placed below 25 feet from the proposed ground surface should be compacted to 95 percent of the maximum dry density. Additionally, all granular fill soils should be compacted to a minimum of 93 percent of the maximum dry density. It is anticipated that due to the granular nature of the proposed fill soils settlement will occur during grading. However, surface monuments should be installed and monitored, and construction should not commence until primary settlement is completed.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

The subject site was evaluated from a geotechnical standpoint for the proposed development as described herein. The site may be developed as proposed; however, geotechnical recommendations provided in this report should be incorporated in the design and construction of the project.

GEOTECHNICAL SEISMIC DESIGN

Active faults as identified by the State are not present on-site nor is the site within an Alquist-Priolo Earthquake Fault Zone (formerly Special Studies Zone). Nevertheless, the site is within a seismically active region prone to occasional damaging earthquakes. The site lies approximately within 2.3 kilometers of two Type B faults (Holser/San Gabriel). Therefore, as a minimum, any structures should be designed per the 2001 California Building Code (CBC). Per the CBC, earthquake loads shall be determined in accordance with Chapter 16A, Division IV. The parameters provided in the following table are from the 2001 California Building Code Chapter 16A.

The purpose of the CBC earthquake provisions is primarily to safeguard against major structural failures and loss of life, not to limit damage or maintain function. Therefore, the values provided in the CBC should be considered minimum design values. Cracking of walls and possible structural damage should be anticipated in a significant seismic event.

CBC – CHAPTER 16A TABLE NO.	SEISMIC PARAMETER	VALUE PER 2001 CBC
16 – I	Seismic Zone Factor, Z	0.40
16 – J	Soil Profile Type	S_D
16 – Q	Seismic Coefficient, C_a	.44*Na*
16 – R	Seismic Coefficient, C_v	.64*Nv*
16 – S	Near-Source Acceleration Factor, N_a	1.27
16 – T	Near-Source Velocity Factor, N_v	1.56
16 – U	Seismic Source Type (fault, distance)	B (San Gabriel & Holser, 2.3 km)

SITE PREPARATION AND GRADING

General

Based on a review of the proposed grading plan, the subject site will involve substantial grading with cut and fill slopes proposed to achieve design grades. Remedial grading will be required to prepare the site for the proposed development. All aspects of grading including site preparation, excavation, and fill placement should per the City of Santa Clarita Grading Ordinance.

Site Cleanup

Areas to be graded should be stripped of vegetation, trash and debris prior to starting earthwork. All brush and/or tree roots over one-half inch diameter should be removed from cut or fill areas. Minor roots may be blended with the soils during processing as per the approval of the city grading inspector and the project geotechnical consultant.

Buttress/Stabilization

The proposed cut slope on the north side of Dockweiler Drive in the area of Geotechnical Cross Sections 4-4’and 6-6’ requires stabilization. Construction of the recommended buttress will improve the stability of the slope during the design seismic event. The buttress keyway should be approximately 10 feet deep measured from the toe and 40 feet wide and should extend laterally across the extent of the slope as shown on the attached Geotechnical Map. The backcut of the buttress should extend up at a 2(h):1(v) gradient. Geotechnical Cross Sections 4-4' and 6-6’ depict the recommended buttress dimensions.

Buttress Construction

Buttress fills should consist of soils having minimum soil strength of 250 psf and 30 degrees for cohesion and friction angle, respectively. Some of the onsite materials should meet these criteria. The fills should be constructed as soon as possible after the backcut and keyway (base) are excavated. Backfilling should commence the same day that the excavation is completed and the construction should be completed as expeditiously as possible.

All keyways and backcuts should be observed by the project engineering geologist prior to placing any fill. The fill should be placed per the "Fill Placement" section of this report. The face of the slope should be treated as a fill slope.

Buttress Backdrains

Backdrains should be constructed at the heel of all keyways and behind the buttress at maximum vertical intervals of 25 feet or at each terrace interval. The elevation of the drains should be adjusted so that the outlet pipes (nonperforated) may be installed with a minimum 1% fall. The backdrains should consist of a perforated minimum 4 inch diameter pipe encased in a minimum 4 cubic feet of drain material per lineal foot of pipe. The drain material should consist of 3/4 to 1 inch rock wrapped in filter cloth, similar to that recommended for subdrains. The drains should be coordinated so as to outlet a minimum of 2 feet above the pad grade or drained into a terrace, if possible. Outlets should be provided at 100 foot intervals with a minimum of 2 outlets. Schematic locations are shown on the attached "Typical Stabilization Fill Detail."

The backdrain locations and installation should be verified in the field by the project engineering geologist. After grading is completed around the backdrain outlet, the outlet should be located, protected from future damage, and covered with a screen clamped in place to prevent rodents from burrowing. To protect the outlet, as a minimum it should be extended 1 foot beyond the slope face, encased in one cubic foot of concrete and marked with a steel post.

Soil Removals

Within all construction areas and 5 feet beyond, all artificial fills, topsoils, and colluvial soils should be removed to firm in-place bedrock. The minimum removal should be 5 feet below the existing grade. Based on our subsurface exploration and laboratory testing, soil removals will be required in the incised drainages prior to structural fill placement and construction of the proposed fill slopes. The anticipated depths of removal of unsuitable materials are shown on the attached Geotechnical Map (Plate 1). The removals on the southern side of the site in the vicinity of HS-2 and HS-3 are anticipated to be on the order of 16 feet below the existing grade at the toe of the proposed fill slopes decreasing to approximately 5 feet in the upper reaches of the alluvial valleys. Trenches T-7 and T-8 were excavated where the drill rig could not access and it was determined that 3 feet of removal would be necessary to expose competent underlying materials in this drainage. On the northern side of the hillside in the area of HS-1, removals of artificial fill and unsuitable soils are anticipated to extend approximately 5 feet below the existing ground surface. All of the removal bottoms should be observed by the project geotechnical consultant to evaluate if local areas exist where deeper removals are necessary.

Undercutting

The building pad portions of bedrock cut areas should be overexcavated a minimum 5 feet below finished pad grades and should extend a minimum of 5 feet beyond the building area. In pavement areas, bedrock should be undercut 2 feet. The removed soils may be reused as fill providing they are mixed and blended so that the completed engineered compacted fill pad is uniform with respect to soil classification, moisture content, density and expansion characteristics.

Daylight Pads

The cut portions of the daylight pads within building areas and 5 feet beyond the building perimeters should be undercut at least 5 feet and capped with engineered compacted fill. The differential fill thickness within building areas should be limited to a maximum of 10 feet, which can be accomplished by performing additional cuts adjacent to areas receiving large amounts of fill. The fill cap should consist of soils that are similar to the fill portion of the pad. The differential fill thicknesses should be further evaluated during the 40-scale plan review.

Preparation of Fill Areas

All areas to receive fill should be processed before placing fill. Processing should consist of surface scarification to a minimum depth of 8 inches, moisture conditioning to a minimum of 1% over the optimum moisture content, and recompaction to a minimum of 90% relative compaction. Optimum moisture content and maximum dry density should be determined per ASTM D 1557.

Fill Placement

On-site materials obtained from excavations may be used as fill soils. Fill soils should be free of all deleterious materials including trash, debris, organic matter, and rocks larger than 12 inches. Fill soils should be placed in thin uniform lifts, brought to 1% over the optimum moisture content, and compacted to a minimum of 93% relative compaction. Fills exceeding 25 feet in depth should have the lower portion (fills placed 25 feet below proposed ground surface) compacted to a minimum of 95% relative compaction. The 95% compaction criteria also apply to any fill placed below subdrain elevations. Imported fill soil should be similar to the on-site soil. Sources of import fill should be approved by the project geotechnical consultant.

Keying and Benching

All fills placed on slopes steeper than 5(h):1(v) should be keyed and benched (horizontal benches) into firm competent in-place soil or bedrock (after all required removals are made). All keyways should be a minimum of 15 feet wide and cut a minimum depth of 2 feet at the toe into firm competent in-place materials. Keyways should be tilted into the slope and should be at least 3 feet deep at the heel (measured from below the slope toe elevation). The keyways should be observed by the project geotechnical consultant before placing any fill. Horizontal benches should be a minimum of 5 feet wide, i.e. a minimum 5 feet of competent material. Benching should be observed by the project geotechnical consultant before placing any fill soils. The vertical portion of the bench in firm material should not exceed 5 feet. A typical benching detail is attached.

Hard Rock

In bedrock areas, it should be possible to make the majority of the cuts using standard grading operations. However, it should be anticipated that some oversize rock (in the form of boulder clasts greater than 12 inches diameter may be encountered which may requiring heavy ripping. Clasts that do not break down to less than 12 inches diameter should be placed in approved deeper fill areas 10 feet or more below finished grade and 20 feet or more horizontally from the slope face in accordance with the attached rock disposal detail.

Consideration should be given to undercutting street areas to facilitate installation of utilities.

Subdrains

Subdrains should be placed in tributary ravines and canyon areas which will receive engineered compacted fill exceeding 7 feet in thickness. Suggested locations are shown on the attached Geotechnical Map (Plate 1). The subdrains should be constructed at least 10 feet below ultimate finished grade and constructed as shown on the attached typical subdrain detail. The drains should be installed in a trench cut into competent alluvial or bedrock materials exposed in the canyon cleanouts (see attached canyon cleanout detail). All cleanouts should extend to competent bedrock materials or firm native soils exposed at the toe of the fills slopes and should be observed by the project geotechnical consultant.

The subdrains should be encased in 9 cubic feet of drain material per lineal foot of pipe. The drain material should consist of 3/4 to 1 inch float rock or equivalent wrapped with filter cloth with an equivalent opening size (EOS) ranging from 70 to 100 units (such as Supac 4NP, Mirafi 140S, or equivalent). The pipe should consist of a minimum 6 inch diameter perforated PVC (Schedule 40) pipe or equivalent (such as corrugated slotted pipe). Where two or more 6 inch drains merge, they should be extended with 8 inch diameter piping and where a 6 inch pipe merges with the 8 inch piping, they should be extended with 10 inch piping. Perforations should be no more than ½ inch diameter. The last 10 feet of drain prior to the outlet should be non-perforated. A cutoff wall should be constructed at the transition from perforated to non-perforated pipe.

The subdrain location and installation should be verified in the field by the project engineering geologist. The subdrain outlet at the slope face should be at least 2 feet above the slope toe or 6 inches above any gunited swales. Immediately after grading is completed around the subdrain outlet, the outlet must be relocated, protected from future damage, and covered with a galvanized screen clamped in-place (using a stainless steel clamp) to prevent entry of small animals. To protect the outlet, as a minimum it should be extended 1 foot beyond the slope face, encased in one cubic foot of concrete and marked with a steel post.

The project civil engineer should survey the subdrain and include the information on the as-built map. Outlet locations must be maintained to allow unrestricted flow from the subdrain systems.

Temporary Excavations

Temporary slopes should conform to the requirements of CAL/OSHA. Surcharge loads should be setback sufficient distances from the tops of temporary excavations.

Shrinkage, Bulking, and Subsidence

Shrinkage or bulking is considered to be the volume loss or gain from cut to fill, including soil excavated and recompacted within removal areas. Subsidence is considered to account for stripping of vegetation and densification of the upper 8 inches of surface soils over the site. The shrinking and bulking values presented are based on an assumption that fills will be compacted to an average of 93% of the maximum dry density. The actual in-place compacted density can vary with the type of material compacted, the compactive effort applied, and the moisture content of the soils at the time of compaction. Preliminary estimates of shrinkage, bulking, and subsidence are presented below:

Material	Shrinkage/Bulking*	Ratio: Fill/Cut
Artificial Fill/Colluvium	5-10% Shrinkage	0.90 - 0.95
Alluvium*	0-10% Shrinkage	0.90 - 1.0
Pacoima Formation	0-10% Bulking	1.0 - 1.1
Saugus Formation	0-10% Bulking	1.0 - 1.1

As an example: In the alluvium, 1 cubic yard of cut will produce 0.90 to 1.0 cubic yards of engineered compacted fill.

Subsidence of 0.2 to 0.3 feet is considered for stripping of vegetation and densification of the upper 8 inches of surface soils.

These estimates are provided for planning purposes only. If quantities are critical, the contractor and designer should observe actual quantities during grading to evaluate the necessity of grade changes to achieve planned final grades and balance quantities of cut and fill.

Utility Trench and Interior Slab Backfill

All utility trench backfill within building areas, parking and drive areas, and backfill beneath interior slabs should be compacted to at least 93% of the maximum dry density. Any disturbed subgrade soils should also be compacted to at least 93% of the maximum dry density.

Berms

Compacted earthen berms should be constructed on pads adjacent descending slopes to direct water away from the slope or the pads should be positively sloped away from the top of slope.

SLOPE CONSTRUCTION

General

Manufactured cut and fill slopes as shown on the Geotechnical Map, Plate 1, will be constructed at maximum gradients of 2(h):1(v).

Fill Slopes

The proposed fill slopes should be keyed and benched into competent soil or bedrock as described above. Where possible, outer slope faces should be overfilled and trimmed back to provide for firm, well compacted surfaces. If it is not possible to overfill the slopes, it will be necessary to sheepsfoot and/or grid roll the slopes. The slope faces should be tested and reworked as necessary to achieve the 93 percent compaction required.

Some select grading may be required when placing fill materials within 10 feet of slope faces. Fill soils near slope faces should have enough fines to develop at least 250 psf cohesive shear strength for 2(h):1(v) maximum gradient slopes. This is a minimum strength requirement based on stability calculations to provide for protection against surface erosion and surficial slope instability.

Cut Slopes

In addition to the recommended buttress in the area of Section 4-4', all other cut slopes should be observed by the project geologist to evaluate if adverse geologic conditions are present. Slopes exposing adverse conditions may require stabilization, buttressing, or slope laybacks. Cut slopes will likely require a stabilization fill.

Slope Maintenance

Slopes constructed within the site will require maintenance or protection to reduce the risk of erosion and degradation with time due to natural or man-made conditions. The slopes should be planted and irrigated as described below.

Slope (fill or soil cut slopes) planting should consist of dense, deep rooting, drought resistant groundcover and possibly shrubs or trees. A reliable irrigation system should be installed, adjusted so that overwatering does not occur, and periodically checked for leakage. Overwatering of slopes can cause expansion, erosion and surficial failures, and must be avoided. A uniform, near optimum moisture content should be maintained below the slope surface. The slopes should not be overwatered and should not be watered before forecasted rain. All drainage structures should be kept in good condition and clean. Burrowing animals (e.g., ground squirrels) can destroy slopes; therefore, where present, immediate measures should be taken to eliminate them.

SOIL EXPANSIVENESS

The subject site is considered to be underlain by soils having a very low to low expansion potential (Expansion Index of less than 50). Near surface soils, such as artificial fill, alluvium or colluvium may have higher expansions in the 51-90 expansion index range. These materials, however, are not expected to make up the majority of the surface soils at the conclusion of grading. Therefore, within each graded pad, the upper 5 feet of subgrade materials should be uniform in its soil expansion characteristics. The actual expansion potential of the upper soils should be determined at the conclusion of the grading operation on a lot by lot basis.

Expansive soils contain clay minerals that change in volume (shrink or swell) due to variations in soil moisture content. The amount of volume change depends upon soil swell potential, availability of water, and soil restraining pressure. Geotechnical recommendations presented herein are generally consistent with the standard level of practice in this area. However, they are not intended to completely eliminate the effects of expansive soils. The following recommendations are provided for construction on expansive soils.

a) Positive drainage away from structures should be provided and maintained and should not be changed creating an adverse drainage condition. Ponding or trapping of water in localized areas adjacent foundations can cause differential moisture levels in subsurface soils. Plumbing leaks should be immediately repaired.

b) Initial landscaping should be undertaken in unpaved areas adjacent structures. However, trees and shrubbery should not be planted where roots can grow under foundations.

c) Landscape watering should be held to a minimum; however, landscaped areas should be maintained in a uniform moist condition.

FOUNDATION RECOMMENDATIONS

The proposed structures may be supported by conventional concrete foundations in accordance with the recommendations presented below. The proposed chapel, retaining walls and accessory structures on the north side of Dockweiler Drive will be constructed primarily in bedrock cut areas. The proposed residential structures on the south side of Dockweiler Drive will be constructed in both cut and fill areas. Cut areas will expose primarily Saugus Formation. Fill areas will most likely be composed of engineered compacted fill placed over bedrock. For preliminary foundation design, the following recommendations are for non-expansive soils. The expansion potential of the subgrade soils should be reevaluated following the completion of grading.

Shallow Foundations

Shallow footings should be at least 12 inches wide and embedded at least 18 inches into firm native soils or certified compacted fill soils. Footings of minimum size and embedment, placed in firm competent materials may be designed for a maximum bearing capacity of 1,500 pounds per square foot (psf). The bearing capacity may be increased by 300 psf for each additional foot increase in footing width and by 500 psf for each additional increase in footing depth, up to a maximum bearing capacity of 3,000 psf. The allowable bearing capacity may be increased by one-third for short term loads associated with wind and seismic loading.

Footing reinforcement should be in accordance with the structural engineer’s recommendations but should be a minimum of 1-#4 top and bottom.

Settlements of shallow foundations will essentially occur due to elastic compression of the supporting soils, and should be complete shortly after the loads are applied. Maximum settlement of a properly designed and constructed foundation should be less than ½ inch.

All footing excavations should be approved by the geotechnical engineer prior to placing reinforcing steel. Any loose or soft soils exposed in the excavation should be over excavated and the area backfilled with fill compacted to at least 93 percent compaction, as directed by the geotechnical engineer.

Foundations on or near slopes that are sensitive to differential movement should be deepened to provide setback to the slope face. The minimum setback should be per Chapter 18 of the California Building Code (H/3 feet where H is equal to the slope height) with a minimum 5 foot horizontal setback. Where loose soil exists on a slope, additional setback distances will be required. The setback should be measured from the existing slope discounting any loose soil on the slope. Accessory structures, such as concrete walkways, that are sensitive to differential movement should be supported by foundations meeting the setback criteria. A structure such as a fence near the descending slopes not meeting the setback requirements could move laterally.

Lateral Force Resistance

Lateral forces may be resisted by passive earth pressure and friction. Passive earth pressure in competent native soil or compacted fill may be taken as an equivalent fluid having a density of 300 pounds per cubic foot (pcf), where the adjacent ground surface is horizontal. The passive pressure for footings located within 5 feet of the face of a descending slope should be designed using 200 pcf. The friction between the bottom of the footings and the supporting soils may be taken as 0.5. These are ultimate values and do not contain a factor of safety. These passive pressure values may be used for design of isolated flagpole footings.

The friction and passive resistance may be combined for design, provided the combined value is reduced by one-half. These values also apply for short-term wind or seismic loads.

Slabs-on-Grade

For non-expansive soils, slabs-on-grade should be a minimum of 4 inches thick and reinforced with No. 4 bars spaced 24 inches on-center each way. Concrete slabs should be underlain by 10-mil visqueen which is overlain by 2 inches of clean sand or aggregate base.

The upper 12 inches of subgrade soils in slab areas should be scarified, brought to near optimum moisture content and compacted to a minimum of 93 percent relative compaction prior to the placement of concrete. Drainage away from the edges of the concrete flatwork should be maintained at all times. Water should not be allowed to pond adjacent to the concrete.

Where a slab or walkway is within 2 feet of the top of a slope, the slope side of the slab/walk should be equipped with a minimum 6" wide by 12" deepened edge. A deepened perimeter edge should also be considered on all slabs where surface water could infiltrate the subsurface layers. The perimeter edge should extend a minimum of 8 inches below the bottom of the slab and have a width of 6 inches. Reinforcement in deepened edges should be continuous with the slab reinforcement or doweled into the slab.

Concrete Cracking

Concrete shrinks as it cures resulting in shrinkage tension within the concrete mass. Since concrete is weak in tension the development of tension results in cracks within the concrete. Therefore, the concrete should be placed using procedures to minimize the cracking within the slab. Shrinkage cracks can become excessive if water is added to the concrete above the allowable limit and proper finishing and curing practices are not followed. Concrete mixing, placement, finishing, and curing should be performed per the American Concrete Institute *Guide for Concrete Floor and Slab Construction* (ACI 302.1R-89). The concrete slump should be per the structural engineer's specifications for concrete slabs-on-grade. Where shrinkage cracks would be unsightly, concrete slabs on grade should be provided with tooled crack control joints at about 10 to 15 foot centers or as specified by the structural engineer.

Tile Flooring

Tile flooring can crack, reflecting cracks in the concrete slab below the tile. Therefore, if tile flooring is used, this should be considered in the design of concrete slabs-on-grade where tile will be placed. The tile installer should consider installation methods that reduce possible tile cracking. Placement of a vinyl crack isolation membrane between tile and concrete slabs on-grade (utilizing approved materials and techniques per Tile Council of America/Ceramic Tile Institute guidelines) is one such method to reduce potential tile cracking.

SOIL CORROSIVITY

Three bedrock samples were tested for corrosivity by an independent testing laboratory. The results of the testing indicate that on-site bedrock is negligible for sulfate and chloride exposure. The testing did indicate that for steel and other buried ferrous metals, the on-site bedrock may be moderately corrosive and to copper piping, the bedrock materials would be corrosive due to the presence of nitrogen and ammonia in the soils. Test results are included in Appendix B.

RETAINING WALLS

General

Retaining walls may be constructed within the subject site provided the following recommendations are followed.

Lateral pressures exerted on retaining walls at the subject site by backfill material will depend on the type of backfill materials used (gravel, clay, etc.), and the ability of the wall to deflect or rotate at the top. Cantilever type retaining walls that are capable of deflecting and/or rotating may be designed for active pressure conditions. However, restrained retaining walls should be designed for at-rest earth pressures, which are higher than the active pressure.

Additional pressure may also be exerted on retaining walls besides the backfill pressure. For example, foundations located behind retaining walls will laterally surcharge the retaining wall. Furthermore, sloping backfill materials will exert more lateral pressure on retaining walls than level backfill.

Geotechnical recommendations for retaining wall design are provided below. All backfill materials are assumed to be non-expansive granular materials.

Retaining Wall Foundations

Continuous footings founded on level ground may be designed to impose a uniform allowable soil bearing pressure of 1,500 psf. Footings should be embedded a minimum of 18 inches into engineered compacted fill and should have a minimum width of 24 inches. Footing reinforcement should be per the structural engineer's recommendations.

All footings should be cut square and level and cleaned of all loose slough prior to casting concrete. Soil excavated from the footing trenches should not be spread over any areas of construction unless properly compacted. The footing excavations must be observed by the project geotechnical consultant prior to placing reinforcing steel. The footings should be cast as soon as possible to avoid deterioration of the footing subsoils.

Active Pressures

Retaining walls that are fully restrained and can not deflect should be designed for an at-rest earth pressure equivalent to a fluid having a density of 45 pcf and 55 pcf for level and 2(h):1(v) sloping backfill, respectively. Retaining walls that are capable of partially deflecting should be designed for an active earth pressure of 35 pcf and 45 pcf for level and 2(h):1(v) sloping backfill, respectively. Maximum backfill slope angles are 2(h):1(v).

Foundations behind the retaining walls should be deepened to below a 2(h):1(v) slope up from the lower retaining wall foundation level. Alternatively, the retaining wall should be designed for the additional surcharge pressures.

Since the site is located in an active seismic area, retaining walls are expected to experience additional surcharge pressure due to backfill inertia. Walls greater than 10 feet in height should be designed for a seismic lateral pressure taken as an inverted triangular pressure of 20 pcf. The resultant of the seismic pressure should be considered to act 0.67H above the base of the wall, where H is the height of the wall measured from the base of the footing to the top of the backfill.

Lateral Force Resistance

Lateral forces may be resisted by the passive earth pressure and friction values presented above for building foundations.

Retaining Wall Drainage and Backfill

Retaining walls should be provided with a drainage system to mitigate possible buildup of hydrostatic pressures. Drains should consist of granular material placed behind the walls drained by either weep holes or perforated pipe.

Drain pipes should consist of 4" diameter perforated PVC (Schedule 40) or equivalent, established behind the walls. The drainage system should consist of a minimum 1 foot wide zone of clean, ¾" crushed rock placed along the base of the wall.

To prevent moist walls, the entire wall should be waterproofed and the drainage system should extend to within 2 feet of the finished grade and capped with 1 to 2 feet of compacted on-site soils. Perforated backdrain pipes should connect to non-perforated 4" diameter PVC pipes (Schedule 40) that are sloped a minimum of 1% to an approved outlet. The invert of the drainage pipe should be at least 6 inches below any interior floor slab.

Weep holes may be used in lieu of perforated pipes for exterior walls. A one-foot square section of crushed rock and ¾" gravel should be placed continuously along the base of the wall behind the weep holes.

On-site granular soil may be used for backfill material. The backfill should be placed in 6 to 8 inch lifts, at near optimum moisture content, and compacted to at least 90% of the maximum dry density. Light equipment should be used immediately behind walls to prevent possible overstressing or excessive tilting.

SITE DRAINAGE

Positive drainage should be provided away from structures during and after construction. Planters near a structure should be constructed so irrigation water will not saturate footing and slab subgrade soils. The pad must be graded at a minimum gradient of 2 percent for landscaped areas away from all structures to an approved drainage course. Drainage water should not be allowed to gather or pond against foundations or building pads. Trees and shrubbery should not be planted where roots can grow under foundations.

GUTTERS AND DOWNSPOUTS

Gutters and downspouts should be installed to collect roof water that may otherwise infiltrate the soils adjacent the structures. The downspouts should be drained into PVC collector pipes (or other positive drainage) that will carry water away from buildings.

EXTERIOR SLABS AND WALKWAYS

All exterior concrete slabs-on-grade and walkways should be a minimum of 4 inches thick. For concrete driveways (auto traffic only), the thickness should be increased to a minimum of 5 inches. The slabs should be reinforced with a minimum of #3 bars on 24 inch centers each way. Walkways may be unreinforced provided they are cut into square panels.

All planter areas should be graded so excess water drains onto, rather than beneath, adjacent concrete flatwork.

Concrete slabs on grade should be provided with tooled crack control joints at suggested 10 to 15 foot centers. Concrete shrinkage cracks will become excessive if excess water is added to the concrete, and proper finishing and curing practices are not followed. Finishing and curing should be performed per the Portland Cement Association Guidelines. The concrete slump should not exceed 4 inches.

PARKING AND DRIVE AREA STRUCTURAL SECTIONS

The following pavement structural sections were developed for preliminary planning purposes. The following estimated pavement structural sections for the subject development are based on anticipated “R” Values for the soil types encountered during our site investigation. A composite sample of bedrock/soils representative of the cut/fill materials expected in the pavement areas was tested. The “R” value of the composite sample was 71. There is, however, the possibility that silt and clay beds may be exposed in some cut areas, such as the Chapel parking lot. An “R” value around 15 to 20 would be expected for this material. Preliminary pavement sections for “R”-values of 15 and 45 are presented below. Actual traffic indices for each paved area should be confirmed and final “R” values should be obtained at the conclusion of grading. All planter areas should be graded so excess water drains onto and not beneath the adjacent AC pavement and curbs.

PRELIMINARY PAVEMENT SECTIONS		
Assumed Traffic Index	**Recommended Structural Section for "R" Value = 15**	**Recommended Structural Section for "R" Value > 45**
4.5	3" AC / 7" AB	3" AC / 4" AB
5.0	3" AC / 9" AB	3" AC / 4" AB
6.0	3" AC / 12" AB or 4" AC / 10" AB	3" AC / 6" AB
7.0	3" AC / 15" AB or 4" AC / 13" AB	3" AC / 8" AB or 4" AC / 6" AB
AC = Asphaltic Concrete AB = Class II Aggregate Base or Similar		

Subgrade Preparation

The subgrade soils within areas of proposed paving should be moistened to slightly above the optimum moisture content and compacted to at least 90% (93% for granular soils) of the laboratory standard prior to placing aggregate base.

Aggregate Base Preparation

The aggregate base materials should be moistened to slightly above the optimum moisture content and compacted to at least 95% of the laboratory standard prior to placing concrete.

PLAN REVIEW

We recommend that grading and foundation plans and specifications be provided to our office for review prior to construction in order to confirm that the full intent of the recommendations presented herein have been applied to the project design.

CLOSURE

This report was prepared within the scope of generally accepted geotechnical engineering practices under the direction of a licensed geotechnical engineer and certified engineering geologist. No warranty, express or implied, is made as to conclusions and professional advice included in this report. Gorian and Associates, Inc. disclaims responsibility and liability for problems that may occur if recommendations presented herein are not followed.

This report was prepared for the owner and his design consultants solely for design and construction of the project described herein. It may not contain sufficient information for other uses or the purposes of other parties. These recommendations should not be extrapolated to other areas or used for other facilities without consulting Gorian and Associates, Inc.

Recommendations herein are based on interpretations of the subsurface conditions concluded from information gained from subsurface explorations and a surficial site reconnaissance. The interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the site. Therefore, persons using this report for bidding or construction purposes should perform such independent investigations as they deem necessary.

Grading and foundation work at the site should be performed per the current City of Santa Clarita Building Code. Due to possible subsurface variations, all aspects of field construction addressed in this report should be observed by the project geotechnical consultant. Services of the geotechnical consultant should not be construed to relieve the owner of contractors of their responsibilities or liabilities.

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Thank you for the opportunity to submit this geotechnical report. Please do not hesitate to call if you have any questions regarding this report.

Respectfully submitted,

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REFERENCES

Blake, T.F., 1999, EQFAULT, a computer program for the deterministic prediction of peak horizontal acceleration from digitized California faults. unpublished private program. Updated 1991, 1993.

Blake, T.F., 2000, Preliminary fault-data for EQFAULT and FRISKSP, unpublished support software support information.

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, 1999, Oakland, California, California Strong Ground Motion Instrumentation Program, Sacramento, California, p. 23-49.

California Department of Conservation, Division of Mines and Geology, (1987), Landslide Hazards in the East Half of the Newhall Quadrangle, Los Angeles County, California. Landslide Hazard Identification Map #7, DMG Open-File Report 86-16.

California Division of Mines and Geology (CDMG), 1997, Guidelines for evaluating and mitigating seismic hazards in California, Special Publication 117, 74 pp.

California Division of Mines and Geology, 1997, Seismic Hazard Evaluation of the Newhall 7.5-Minute Quadrangle, Los Angeles County, California, Open-File Report 97-11.

California Division of Mines and Geology, 1998, State of California Seismic Hazard Zones – Mint Canyon Quadrangle Official Map. Released February 1, 1998.

Cetin, K.O., Seed, R.B., Der Kiureghian, A., Tokimatsu, K., Harder, L.F., Jr., Kayen, R.E., and Moss, R.E.S., 2004, Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential: ASCE Journal of Geotechnical and Geoenvironmental Engineering, v. 130, n. 12, p. 1314-1340.

Chang, S.W., Bray, J.D., and Seed, R.B., 1994, Ground Motions and Local Site Effects. in Stewart, J.P., Bray, J.D., Seed, R.B. and Sitar, N. eds., Preliminary report on the principal geotechnical aspects of the January 17, 1994 Northridge earthquake, Earthquake Engineering Research Center, University of California at Berkeley, Report No. UBC/EERC-94/08.

Dibblee, Thomas W., Jr., (1996), Geologic Map of the Newhall Quadrangle, Los Angeles County, California.
Dibblee Geological Foundation Map #DF-56.

Earth Systems (Southern California), (2005), Master Plan Update, The Master's College, 21726 Placerita Canyon Road, Santa Clarita, Los Angeles County, California. PL-05219-10, dated February 25.

Gorian and Associates, Inc., (2005), Foundation and Seismic Design Recommendations, Proposed MacArthur Chapel, Master's College, Santa Clarita, California. Work Order: 2668-0-0-10, Log Number: 23993, dated August 3.

Hart, E.W. and Bryant, W.A., 1999, Fault rupture hazard zones in California. California Division of Mines and Geology Special Publication 42 (revised).

Idriss, I.M., and Boulanger, R.W., 2004, Semi-empirical procedures for evaluating liquefaction potential during earthquakes; Proceedings of the 11th SDEE and 3rd ICEGE, University of California, Berkeley, January 2004, plenary session, p. 32-56.

Pradel, D., 1998, "Procedure to Evaluate Earthquake Induced Settlements in dry Sandy Soils," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 124, No. 4, April.

REFERENCES (cont.)

South Coast Geological Society, (1982), Geology and Mineral Wealth of the California Transverse Ranges. *Age of Upper Saugus Formation at Newhall, California and Implications as to the Age of the Santa Susana Mountains,* page 330, by Jerome Treiman.

Southern California Earthquake Center, 1999, “Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction in California.”

APPENDIX A

LOGS OF SUBSURFACE DATA

APPENDIX B

LABORATORY TESTING

General
Recent laboratory test results on selected relatively undisturbed and bulk samples are presented below. Tests were performed to evaluate the physical and engineering properties of the encountered soils, including field moisture and density, compaction characteristics, expansion/consolidation potential, shear strength, and grain size distribution.

Field Density and Moisture Tests
In-situ dry density and moisture content were determined for relatively undisturbed samples obtained from the exploratory excavations. The test results and a detailed description of the soils encountered are shown on the attached logs.

Optimum Moisture-Maximum Density Curve
Maximum density/optimum moisture tests (compaction characteristics) were performed on selected bulk samples of the encountered materials in general accordance with ASTM test method D1557. The results are as follows:

Boring	Depth (feet)	Visual Soil Classification	Maximum Dry Density – pcf	Optimum Moisture Content - %
B-1	14	Yellowish brown clayey silt	114.0	15.0
B-2	18	Yellowish brown f/c sand	133.0	8.5
B-3	5	Yellow silty fine sand	119.0	11.0
B-4	9	Yellowish brown sandy silt	120.0	13.5

Expansion Test
Selected samples of the encountered soils were tested for expansiveness. The samples were passed through the #10 sieve, wet to approximately 80% of the optimum moisture content, and compacted in a one inch thick ring. An axial load of 144 psf was applied to the sample and water was added to saturate the sample. Twenty-four hours after adding water, the amount of expansion was evaluated in terms of the expansion index. The results are as follows:

Boring	Depth (feet)	Visual Soil Classification	Expansion Index	Index Range
B-1	3	Yellowish brown clayey fine to coarse sand	47	21-50
B-10	1	Brown clayey to silty fine to coarse sand	10	0-20

Direct Shear Test
Strain controlled direct shear testing was performed on twelve relatively undisturbed samples and four remolded samples of the earth materials encountered during our exploratory program. The sample sets were saturated prior to shearing under axial loads ranging from 1175 to 4700 psf at a rate of 0.02 inches

per minute. The ultimate shear strength results are attached as graphic summaries on Figures B.1 through B.16.

Residual Shear Tests

Residual direct shear testing was performed on one undisturbed sample. In each case, the shear plane was saturated prior to shearing under axial loads of 2,350 and 4,750 psf. Each sample was repeatedly sheared until similar shear strength was observed. This process included an initial cycle to establish a shear plane after which the specimen and testing apparatus were reassembled for subsequent cycles. Following each run of a shear test, the top half of the specimen was separated from the lower half with continued forward motion and then the apparatus and specimen were reassembled for a subsequent cycle. Between cycles the specimen is allowed sit under the axial loads for approximately 5 minutes. The results of the tests are shown as graphic summaries on Figure B.17.

Load Consolidation Test

Load consolidation tests were conducted on ten relatively undisturbed and four remolded samples of the materials encountered. Test loads were added in increments to a maximum of 8,000 psf or 18,800 psf. Water was added at axial loads of 1,000 or 2,000 or 9,400 psf to study the effects of moisture. The results are attached as graphic summaries on Figures B.18 through B.31.

Grain-Size Distribution

Grain size distribution analyses were performed on two alluvial samples. The grain size was evaluated using a hydrometer. Hydrometer analyses were performed using a 50 gram sample. The sample was soaked overnight in a solution consisting of distilled water and a water softener. After soaking, distilled water was added to the sample, it was thoroughly agitated in a 1000 mg flask, and a hydrometer was placed in the flask. At selected time intervals, the hydrometer level in the solution was measured to evaluate the particle suspension. From the readings taken, the percentage sand, percentage silt, and percentage clay was evaluated. The percentage of clay was assumed to be all particles finer than 0.005 mm. The results are presented below.

Boring	Depth (feet)	% Sand	% Silt	% Clay	Classification
HS-2	14	64	19	17	Silty sand
HS-3	9	79	12	9	Silty sand

APPENDIX C

EQ FAULT DATA

APPENDIX D

SLOPE STABILITY ANALYSES

As discussed in the body of this report, the proposed cut slope adjacent to the proposed parking area on the north side of Dockweiler Drive has a potential for slope instability due to adverse geologic structure (typically north/northwest dipping planes of weakness). The proposed grading near or at the base of the cut slopes may expose these planes of weakness and create an adverse slope stability condition.

Geotechnical sections have been prepared within and adjacent to proposed development areas. A discussion of each of the geotechnical sections analyzed, results of the stability analyses, and proposed remedial grading solutions, if necessary, are presented below.

The material strengths for the bedrock, fill and alluvium were developed using information from our laboratory direct shear testing of both undisturbed and remolded samples. The undisturbed samples were saturated and sheared to develop cross-bedding strengths. One undisturbed sample representing the weaker interbeds was saturated, precut along the plane to be sheared, and sheared repeatedly to develop along-bedding strengths. The strengths used in our slope stability analyses are provided below. Ultimate strength parameters were used in completing static analyses and peak strength parameters were used to complete pseudo-static analyses.

SOIL TYPE	UNIT WEIGHT (pcf)	ULTIMATE VALUES		PEAK VALUES	
		COHESION (psf)	FRICTION (degrees)	COHESION (psf)	FRICTION (degrees)
Saugus Formation – across bedding	130	200	37	200	40
Saugus Formation – along bedding (clay)	130	150	17	150	17
Pacoima Formation	130	100	37	100	37
Alluvium	120	250	35	250	35
Engineered Fill	125	300	30	300	30

Analyses were performed for both static and earthquake conditions. Earthquake conditions were simulated by performing a pseudo-static analysis with a lateral coefficient of 0.15g.

Groundwater conditions are typically crucial to the performance of all slopes. Therefore, we have considered it prudent to model these slopes as if some form of seepage may be present. In most cases, we have included two feet of pore pressure (125 psf) along the postulated planar failure surfaces to simulate two feet of water buildup along each surface. For rotational analyses, no water was used in the analysis. It is noted below where water conditions in the analyses differed.

Planar failures were evaluated utilizing Janbu's method. This method divides the postulated failure mass into a series of slices. Interslice forces are not taken into account. The analyses utilized anisotropic soil parameters for Saugus Formation bedrock where bedding was present. Numerous trial surfaces were analyzed for each section. The stability of slopes is commonly stated in terms of the slopes calculated factor of safety. Stability results are listed below and analyses are shown on the attached calculation sheets. The generally accepted lower limit for factor of safety is 1.5 and 1.1 for static and pseudo-static conditions, respectively. Where calculated factors of safety are less than the accepted lower limit, remedial measures were analyzed.

Section 2-2'

Section 2-2' is located on the north side of Dockweiler Drive and represents the highest fill slope. A rotational analysis was completed for the proposed fill slope. In addition, the bedrock supporting the fill has shallow bedding angles that were evaluated for planar type failures. The results of the rotational analysis indicate factors of safety of 1.68 and 1.15. The critical rotational failure plane was located entirely within the engineered fill. Potential planar failures were also evaluated along specific beds observed in boring B-6 at approximate depths of 72 and 80 feet. For each analysis, anisotropic strengths were entered for the Saugus Formation, specifying along bedding strengths for surfaces at angles ranging from 0 to 8 degrees. For surfaces greater than 8 degrees, across bedding strengths were used. The resulting factors of safety were greater than 1.5 and 1.1 for static and pseudostatic, respectively and are summarized in the table below.

Data Filename	Description	Factor of Safety
26682AR	Static, Rotational	1.68
26682ARQ	Pseudostatic, Rotational	1.15
26682CUR	Static, Planar, Upper Bed	2.23
26682CUQ	Pseudostatic, Planar, Upper Bed	1.55
26682CLR	Static, Planar, Lower Bed	1.68
26682CLQ	Pseudostatic, Planar, Lower Bed	1.14

Section 4-4'

Section 4-4' represents the highest cut slope within the proposed development. This area of the site is underlain by a sliver of Pacoima Formation over Saugus Formation with north to northwest dipping bedding at very shallow angles (0 to 8 degrees). A static planar analysis on the slope indicates satisfactory factors of safety; however, the pseudo-static factor of safety was less than 1.1. Critical failure surfaces with unacceptable factors of safety were located near the toe of the proposed slope and near the first bench in slope, as shown on the Geotechnical Map. To obtain a satisfactory factor of safety, a buttress was created that was 10 feet deep, measured from the toe of the slope and 40 feet wide. The backcut needed to maintain the necessary fill thickness in front of the cut is 2(h):1(v) as shown on the cross section. With the buttress in place, analyses were performed to check for potential failures through the buttress and below. The resulting pseudostatic factor of safety was 1.1. The analyses performed on the slope are summarized in the table below.

Data Filename	Description	Factor of Safety
26684AR	Static, Planar, No Stabilization	1.53
26684ARQ	Pseudostatic, Planar, No Stabilization	0.99
26684DR	Static, Planar, Failure Through Buttress	1.65
26684DRQ	Pseudostatic, Planar, Failure Through Buttress	1.11
26684FR	Static, Planar, Failure Beneath Buttress	1.64
26684FRQ	Pseusostatic, Planar, Failure Beneath Buttress	1.11

Work Order: 2668-0-0-10
Log Number: 23978

APPENDIX E

LIQUEFACTION CALCULATIONS

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