Draft Environmental Impact Report
Volume III - Appendices E - J

South Shores Church Master Plan
City of Dana Point

SCH No. 2009041129

Prepared by

LSA

LSA ASSOCIATES, INC.

September 2014

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Geotechnical, Inc.

Geotechnical Evaluation and Slope Stabilization Design for Environmental Impact Report Purposes, for Proposed New Structures at the South Shores Church, City of Dana Point, California

Volume I

Prepared For:

Mr. GG Kohlhagan

South Shores Church
32712 Crown Valley Parkway
Dana Point, CA 92629

Dated: May 22, 2013

Project No. 10132-01

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LGC
Geotechnical, Inc.

May 20, 2013 Project No. 10132-01

Mr. GG Kohlhagan
South Shores Church
32712 Crown Valley Parkway
Dana Point, CA 92629

Subject: ***Geotechnical Evaluation and Slope Stabilization Design for Environmental Impact Report Purposes, for Proposed New Structures at the South Shores Church, City of Dana Point, California***

In accordance with your request, LGC Geotechnical, Inc. has performed a geotechnical evaluation of subsurface conditions relative to the proposed construction of new structures at the South Shores Church located in the City of Dana Point, California. The proposed site development includes phased construction of four, two-story buildings, associated walls, a parking structure, and a meditation garden. Previous iterations of this report have been submitted and reviewed by the City of Dana Point. This integrated report encompasses our previous findings, conclusions, and recommendations as well as responses to review questions in a stand-alone report. It is intended to provide sufficient geotechnical information and design recommendations, as required for environmental impact report purposes, to show that the project can be successfully developed from a geotechnical point of view. Subsequent, specific design reports will be required prior to actual construction.

Please note that the proposed "Master Plan Alternative" was also considered from a geotechnical perspective within the report in order to present the possible design for review as part of the EIR process. The Master Plan Alternative project can also be successfully developed from a geotechnical point of view.

Should you have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Sincerely,

LGC Geotechnical, Inc.

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1.0 INTRODUCTION

The purpose of this evaluation was to review previous geotechnical data relevant to the South Shores Church property located in the City of Dana Point, California (Site Location Map, Page 4), refine and update the geologic model, and provide geotechnical recommendations for the proposed re-development of the site. During previous geotechnical evaluations of the site, numerous borings and trenches were excavated, logged, tested, and reported. LGC Geotechnical has reviewed the referenced geotechnical reports and drilled two additional borings in order to gain supplemental information and to create a baseline of comparison with borings and trenches previously excavated and logged by others (References, Appendix A). Off-site borings, regional and local geologic maps by others, and interpretations of aerial photographs were incorporated into our geotechnical evaluation. The combination of previously available data and supplemental data has provided detailed characterization of the subsurface conditions that may affect the proposed re-development of the site. Specific geologic features were stratigraphically and structurally correlated between borings and a refined geologic model was created for engineering analysis.

The available suite of subsurface data was geotechnically analyzed with the intent to improve the previously proposed mitigation design. The previous mitigation design involved construction of a replacement fill buttress with significant earthwork grading and construction phasing, in addition to installation of a mechanical stabilization system at the completion of earthwork grading (Nicoll, 2006 through 2008d). A revised plan was desired in order to reduce the complexity of construction and potential impact to surrounding neighborhoods. Also, the overall development plan for the Proposed Master Plan has been reduced in scope at the northeast portion of the project with a scaling back of the previously proposed, stabilized flat area and retaining wall to the east of the proposed Christian Education Buildings. The development plan for the Proposed Master Plan Alternative is even further scaled back in overall scope and square footage of structures and incorporates additional setbacks from the property limits. The combined benefits of a refined geologic model, reduced development, and revised stabilization methods presented herein are anticipated to significantly reduce the level of earthwork grading and construction that was previously required. The intent of this report is to present the refined geologic model and to demonstrate feasibility of construction of the planned re-development project using the stabilization methods presented herein.

1.1 Project Description

The South Shores Church is a hilltop property located on the east side of Crown Valley Parkway, approximately a quarter-mile from its intersection with Pacific Coast Highway, in the City of Dana Point, California, as shown in the Site Location Map (Figure 1, Page 4).

The subject site is bounded at the west by Crown Valley Parkway, at the south by an existing residential community, and at the north by a descending graded cut slope and vacant area within an existing apartment complex. At the east boundary, a large, natural slope descends to a graded area with a portion of a golf course and a bike path near the toe-of-slope. Salt Creek runs through the golf course that is adjacent to and below the site.

The proposed re-development of the subject site will include phased demolition of the existing Preschool, Chapel, and Administration/Fellowship Hall. Ground improvement in the form of mechanical slope stabilization will be undertaken at the northeast portion of the site, and various new buildings and retaining walls will be constructed. New buildings will be constructed to the south and north of the existing Sanctuary, which will remain. The new buildings will consist of a Preschool/Administration Building with a Meditation Garden to the south of the Sanctuary, and two Christian Education Buildings and a Community Life Center to the north of the Sanctuary. The proposed buildings are one- and two-story structures, to be set into gently variable topography with the use of interior and exterior retaining walls. Parking areas and access pathways will be reconfigured with relatively minor cut and fill grading and a second-story parking deck is proposed for a portion of the parking area. Proposed structures, relative to each respective design, are depicted on the Geotechnical Maps, Sheets 1 and 5.

This evaluation includes information pertaining to both the Proposed Master Plan and the Proposed Master Plan Alternative. The Alternative Design generally represents a significantly lesser footprint of environmental impact in the majority of areas in comparison to the Proposed Master Plan. Per the Alternative Design, the Christian Education Buildings are reduced in size, the retaining wall at the east side of the property is removed, and the Preschool/Administration Building and parking structure become smaller and further set back from the property limits. Additionally, the Community Life Center becomes a smaller, one-story structure and moves slopeward in order to accommodate an increased distance from Crown Valley Parkway. We anticipate that the City's review of the project can be evaluated for both cases with regards to environmental impact, utilizing the information presented herein.

1.2 __Background__

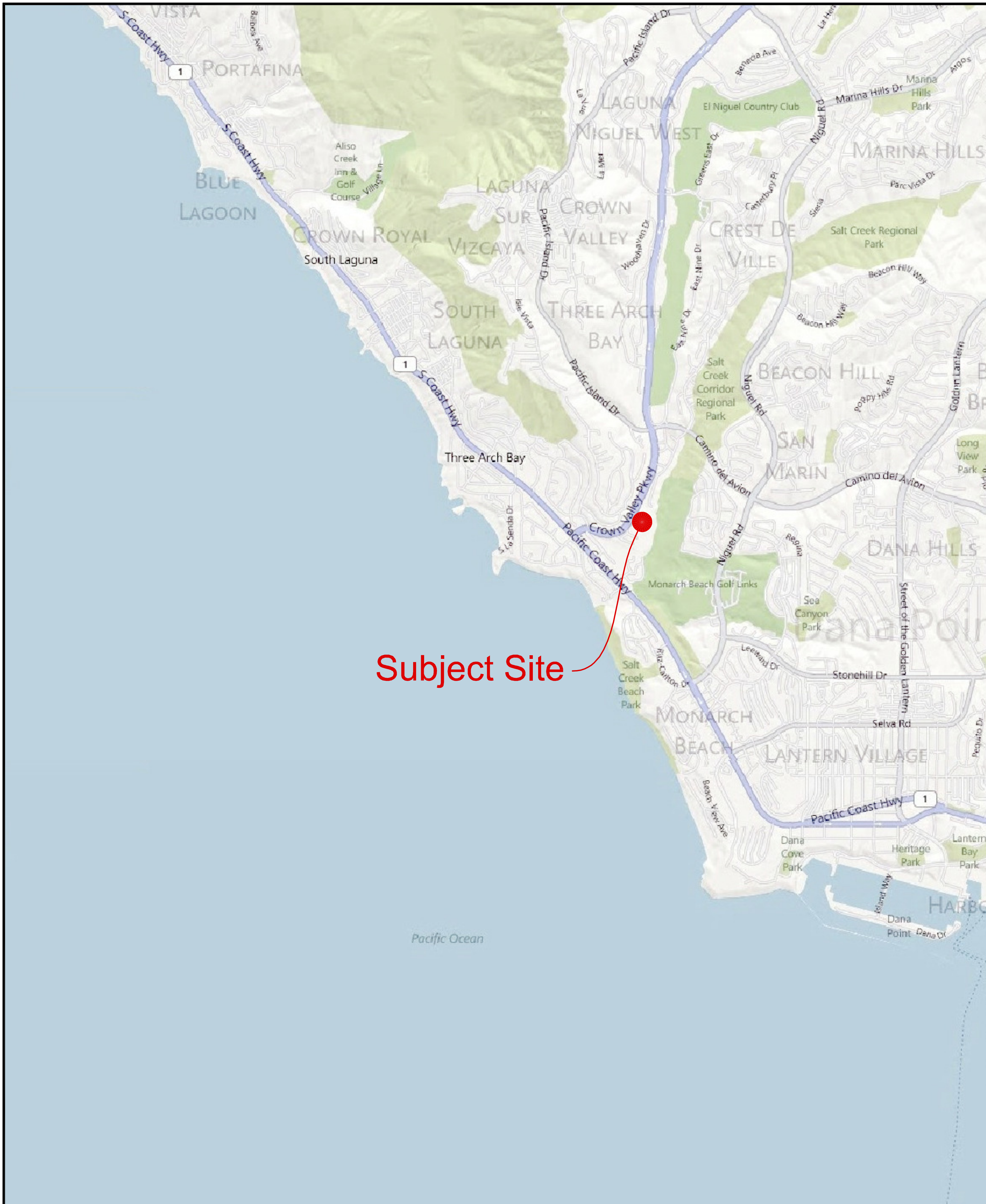
The existing structures at the subject site have been constructed over the many years of existence of the South Shores Church. The existing Sanctuary building is the most modern structure onsite, and it will remain during construction of the proposed improvements. The previous consultant, G.A. Nicoll and Associates, Inc. (Nicoll), provided geotechnical engineering services for the design and construction of the existing crib wall at the southern boundary of the site and Sanctuary (1992 & 1993), and then continued as the geotechnical consultant during the majority of the subsurface investigation that forms the basis for the geologic model presented here.

A series of subsurface investigation and review response reports was provided by Nicoll (References), in support of a previous iteration of the South Shores Church plan. The plan has since been refined, and the geologic model has also been refined based on the subsurface evaluation conducted by LGC Geotechnical that is described below.

1.3 __Subsurface Evaluation__

The recent subsurface evaluation by LGC Geotechnical consisted of the excavation of two large-diameter borings, LGC-1 and LGC-2, at the locations shown on the Geotechnical Maps, Sheets 1 and 6. The purpose of the borings was to obtain additional structural geologic data and to establish a baseline of comparison with previous subsurface excavations by others over the years (References). Previous subsurface investigations both onsite and off-site have been compiled and reviewed, data included herein. Boring and trench locations are depicted on the Geotechnical Maps (Sheets 1 and 6), and boring and trench logs have been included in Appendix B. Results of laboratory testing on samples from recent borings are noted on boring logs and included in Appendix C, Laboratory Test Results.

The combination of the previous investigations and the recent borings by LGC Geotechnical provide a sufficient amount of data for design of mitigation measures for the geotechnical issues that affect the site. Additionally, laboratory testing has been performed by LGC Geotechnical and by others during previous investigations and earthwork activities at the site, and the data will be incorporated into a future grading plan review of the proposed development.



LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	South Shores Church
PROJECT NO.	10132-01
ENG. / GEOL.	TJL / KTM
SCALE	Not to Scale
DATE	May 2013

2.0 GEOTECHNICAL CONDITIONS

2.1 Geologic Structure

The subject site is generally located within the Peninsular Ranges Geomorphic Province, more specifically within the San Joaquin Hills that are located along the southern boundary of the broad Los Angeles Sedimentary Basin. The San Joaquin Hills is an area of coastal uplift estimated to be based on a blind thrust fault at depth. The property is near the top of a hill that is underlain by materials of the Tertiary-age San Onofre Formation, landslide derived from the San Onofre Formation, and artificial fill.

The majority of the subject site is underlain by the San Onofre Breccia, one of the most resilient bedrock formations in South Orange County. The marine sedimentary formation consists of cobble conglomerate zones, cemented zones, and a few zones of well-bedded, fine grained material. The few zones of fine grained material consisting of silt and clay form weaker layers within the otherwise resilient bedrock. Another formational material, the Tertiary Monterey Formation, was identified off-site, near the toe of the large descending slope that underlies the site. The Monterey Formation is primarily a siltstone, and it is known for its potential for landsliding. The two bedrock formations, landslides, and graded areas of artificial fill have altogether created a variable complex of materials at the off-site, toe-of-slope area.

A landslide is present at the northeast portion of the site that follows one of the weak layers of the San Onofre Breccia described above, at depth. A second weak layer at depth below the landslide at the northeast corner of the site was specifically noted by both the previous consultant and LGC Geotechnical as an important geologic control for slope stabilization. Formerly labeled “hypothetical shear” in Nicoll, 2008a, the feature is now labeled “Silty Clay Bed” in this report. The character of the material between the identified landslide and the Silty Clay Bed is variously described as tectonically fractured bedrock and queried landslide. The material below the Silty Clay Bed was observed by LGC Geotechnical to be bedrock.

In general, site data regarding bedding and jointing/fractures can be summarized as follows. Within the formational materials at the site, the fine grained bedding has been interpreted to posses the actual strike and dip of the bedding that underlies the site. Based on review of previous borings and downhole logging observations of a recently excavated large-diameter boring LGC-1, bedding within the coarse grained/cobble beds indicates a large variation of strikes, and a lesser variation of dips. Strike of the coarse grain deposits as measured ranged widely between N85E and N20W, and dips range between 12 degrees south/east and 38 south/east. Fine grain materials are considered to be more representative of actual, originally horizontal bedding. Strike of the fine grain beds generally range between N25W and N10E, while dips range between 12 degrees east and 25 degrees east. More variation is present within the landslide-affected outer slope areas and areas to the south where the east boundary hillside shallows and significantly decreases in height.

In general, within the critical location of areas north of the existing Sanctuary structure, the upper portion of the hillside has a slightly steeper dip range than the lower portion of the hillside indicating a slight synclinal component but with an overall trend close to the character of a dip-slope. The recently excavated boring LGC-2 at the southern portion of the site indicates the bedding there is anomalously southwest-dipping. Fracture orientation was relatively sporadic within the landslide portion of the observed geologic structure, and few fracture attitudes were recorded in previous logs, especially within the predominantly coarse-grained material. Minor shears indicative of tectonic faulting were recorded within various borings, however.

A fault was observed in boring LB-7(B) at a depth of 18 feet, oriented into-slope and within the bedrock core of the site, presented on the Geotechnical Maps (Sheets 1 and 6). The fault is interpreted as a normal fault due to the inclination of the feature and the general extensional regional geologic regime related to uplift (not compression) of the San Joaquin Hills. No geomorphic indicators of the fault were observed in review of aerial photographs. A similarly oriented shear is recorded within nearby boring BA-3. The presence of minor faulting has been considered with relation to the Silty Clay Bed and overall site geologic conditions.

Specific stratigraphic correlation between borings and interpretation of the large suite of available data was necessary for refining the geologic model for geotechnical mitigation of the site relative to the previous consultant's interpretations. The recent boring LGC-1 was advanced at a critical location where previous borings by others had terminated on refusal. Information obtained from the boring was used to compare stratigraphy between previous borings. The Silty Clay Bed observed at 68 feet in depth in LGC-1 was correlated to similarly-described features in older borings and projected to the surface along strike and dip. Previous interpretations did not present the surface location of the feature and did not project the bed to the north and south along bedding.

The surface expression of the Silty Clay Bed was constructed one point at a time, starting with Cross-Sections A-A' and B-B'. Boring BN-1 supports the location of the feature in addition to the information gathered in LGC-1. The total depth of those borings helps to constrain against the presence of additional weak beds at depth. Off-site Boring LB-1(B) behind and below the Silty Clay Bed also helps to constrain against the presence of additional weak beds at depth.

For establishing the location of the Silty Clay Bed in the area of Cross-Section C-C', presence of the fault in LB-7 and the feature at 28.5 feet in depth within Boring BB-106 were important. The fault is interpreted to offset the Silty Clay Bed down to the northwest (normal movement), putting the Silty Clay Bed at the location observed in BB-106. This was supported by a fence diagram constructed through borings BB-106 and BA-1(X) in the area of the existing Sanctuary. The Silty Clay Bed was observed in BB-106 but was not observed in BA-1(X) below the Sanctuary. The feature in Boring BB-104, at 9 feet in depth, established another location of the Silty Clay Bed further to the south in the area of Cross-Section D-D' that lines up with the feature as observed in BB-106.

At the southern portion of the site between the areas of Cross-Sections D-D' and E-E', the descending offsite slope is reduced to a gently-inclined ridgeline. Areas previously graded under the observation and testing of Nicoll (1993) were provided with a stabilization fill and subdrain. The southern boundary of the subject property was provided with a crib wall approximately 215 feet long, backfilled with engineered fill. Recent boring LGC-2 was excavated through the existing engineered fill to evaluate the fill and underlying geologic conditions, as depicted on Cross-Section G-G'. Orientation of bedding is south to southwest in this area, significantly different from the northeast portion of the site. The change in bedding direction may be related to the change in geomorphology of the hillside (reduction in slope height and inclination), as may occur with a resistant anticline within the bedrock. Such an anticline, if present, would not influence the slope stability evaluation of the eastern perimeter slope. The bedding orientation at LGC-2 is geotechnically favorable in that it is into-slope relative to the site's eastern boundary condition.

The Geotechnical Maps, Sheets 1 and 6, present the borings and geologic attitudes of the critical surfaces in each boring depicted with overlays of the Proposed Master Plan and Alternative Design, respectively. The approximate surface location of the Silty Clay Bed is also depicted. Cross Sections A-A' through G-G' depict the interpreted subsurface geologic structure relative to each plan also. Boring logs and trenches from the recent investigation and previous investigations are included in Appendix B for reference.

2.2 *Seismicity and Faulting*

Southern California is an area known for its active faults, and seismic hazards exist for areas of active faulting in the form of ground rupture and ground shaking due to earthquakes. The subject site is not located within an active fault zone, but may still be affected by ground shaking. Some of the active faults that may affect the subject site include the San Andreas Fault, the Newport-Inglewood Fault, and the Whittier Elsinore Fault. The closest significant fault to the site is the active off-shore portion of the Newport-Inglewood Fault Zone, located approximately 3 miles west of the site. The site is located within the San Joaquin Hills; these coastal hills are inferred by indirect evidence to be uplifted along a blind thrust fault at depth.

The subject site is not located within an Alquist-Priolo/Special Studies Earthquake Fault Zone and there are no known active or potentially active faults onsite (CDMG, 2001). Therefore ground rupture due to faulting is not anticipated to affect the site. Secondary hazards from ground shaking are discussed below in the section titled "Geotechnical Hazards".

2.3 *Geologic Material Types*

The following materials were encountered during the recent and previous subsurface investigations. The approximate extent of materials described below is depicted on the Geotechnical Maps and Cross Sections (Sheets 1 through 10).

2.3.1 *Artificial Fill Soils (Map Symbol - Af)*

Artificial fill soils are present across the site with the exception of the central area of the existing parking lot. The maximum depth of fill is estimated to be 25 feet at the southeast portion of the site, placed under the observation and testing of the previous consultant and reported in the referenced grading report (Nicoll, 1993). Boring LGC-2 was recently excavated by LGC Geotechnical for evaluation of the quality of the engineered fill material at the southern portion of the site adjacent to the existing crib wall. The boring log is presented in Appendix B, and laboratory test results are presented on the boring and in Appendix C. Where encountered, the fill was observed to be reddish-brown to dark brown clayey sand with gravel, moist and dense.

2.3.2 Quaternary Landslide (Map Symbol –Qls)

Recent boring LGC-1 was excavated through the upper portion of a landslide at the northeastern portion of the site. At depth, the basal rupture surface of the landslide is estimated to follow one of the weak beds of the San Onofre Breccia or Monterey Formation near the toe-of-slope. The landslide material, where encountered, was highly to moderately weathered cobble breccia and clayey sandstone, moist, and dense.

2.3.3 Tertiary San Onofre Breccia (Map Symbol – Tso)

The primary bedrock formation underlying the site is the San Onofre Breccia Formation. Variable brecciated cobbles and gravels of metamorphic origin are weakly to well cemented within a matrix of clayey sandstone, brown to gray, moist, and very dense. Few, thin beds of clay and silty clay materials were encountered during various phases of subsurface exploration, generally traceable between borings. Also, zones of nested cobbles and boulders were encountered, typically at the base of a coarsening-downward stratigraphic sequence. Correlation of the cobble and boulder zones between borings indicated these high-energy deposits have variable thickness.

The upper, weathered portion of the San Onofre Breccia Formation was observed to be relatively more oxidized, slightly less dense, and weakly cemented in comparison to the same material at depth. There is some question in the recent and previous boring logs and reports as to whether the queried San Onofre Breccia material (Map Symbol - Tso?) on the Geotechnical Map is landslide material or weathered bedrock affected by tectonic shearing. Below the Silty Clay Bed feature, the bedrock in LGC-1 was observed to be fresh, unoxidized, consistently gray, very dense, and weakly to well cemented. Approximate locations of the oxidized to unoxidized bedrock are presented for locations where the contact was encountered in borings at depth or projected, then contoured to match site topography.

2.3.4 Tertiary Monterey Formation (Map Symbol – Tm)

Monterey Formation material is located off-site near the base of the large descending natural slope east of the site. This material generally consists of thinly interbedded siltstone, clayey siltstone, and fine sand lenses, typically brown to dark gray, moist, and stiff to moderately hard in comparison to “soil”, moderately soft in comparison to “rock”.

2.4 Expansion and Corrosion Potential

The expansion potential of the near-surface soils underlying the subject site have been identified by others during construction of the existing improvements as low to moderate based on visual observation. Testing in accordance with ASTM D4829 Test Method indicated site soils possess an expansion index of 78, indicating “moderate” expansion potential (Nicoll, 2006).

Corrosion potential of near surface soils has been evaluated by Nicoll in the referenced report (2007a). Test results indicated that the level of sulfate exposure for concrete is classified as “not applicable”, however, onsite soils are considered very highly corrosive to buried metals (ACI, 2008).

2.5 <u>Geotechnical Hazards</u>

Geotechnical hazards that may affect development of any site include earthquake-induced landslides, liquefaction potential, lateral spreading, subsidence, soil collapse, and potential for tsunami or seiche. Based on review of the Dana Point Seismic Hazards Report (CDMG, 2001), the subject site is located in an area with potential for earthquake-induced landslide, however, the potential hazard to development at the site can be mitigated with implementation of the geotechnical recommendations of this report and future applicable reports.

The site is not located within an area of potential liquefaction (CDMG, 2001), and it is not considered a potential risk for lateral spreading, subsidence, or soil collapse, based on the material types underlying the site, and anticipation that site earthwork will be performed in accordance with project specifications.

The site is not considered to have potential for tsunami or seiche hazard due to the elevation above sea level and lack of a major body of water in the proximity.

2.6 <u>Infiltration Feasibility</u>

Based on the geotechnical conditions encountered during subsurface evaluations by this firm and previous consultants, LGC Geotechnical recommends that no water be purposefully infiltrated to the subsurface on a permanent basis. However, it is our opinion that watering to “mimic ambient rainfall” may be performed for establishment of plantings within the un-improved portions of the site such as the Fuel Management Zone.

Additionally, based on review of the Preliminary Water Quality Management Plan and proposed “bioretention BMPs” planned to be installed adjacent to the proposed buildings, it is our opinion that the planted retention areas will not lead to infiltration of water to the subsurface. The areas are lined with impermeable materials and collected water is ultimately transported to site drainage conveyances (Adam-Streeter, 2012a and 2012b).

2.7 <u>Groundwater</u>

Minor groundwater seepage was encountered sporadically during the subject evaluation and previous evaluations at various depths within deep borings. A static water table was encountered in LGC-1 at approximately 90 feet in depth.

3.0 ENGINEERING ANALYSES

3.1 Soil Shear Strength Parameters

Soil shear strength parameters for the materials that comprise the site, utilized in our slope stability analysis, are provided in Table 1. These values are based upon our experience in the area and review of parameters used by Nicoll, supported by back-calculation of the existing conditions and published shear strength data (References). The back calculations are included in the attached Appendix D, Slope Stability Analyses. The site soil shear strength values were applied to the existing slope in the original condition, without engineered fill at the toe-of-slope, along both the defined landslide rupture surface and the Silty Clay Bed, respectively.

Shear strength values for the controlling feature, the Silty Clay Bed, are the same as the landslide rupture surface shear strength value previously used by Nicoll, reviewed by LGC Geotechnical and accepted for the project. The material noted as Tso(?), on the Geotechnical Maps and Cross Sections has been modeled using shear strength values obtained during direct shear testing of multiple saturated samples taken from the same material interval (Nicoll, 2008), also reviewed and geotechnically accepted for the project.

One additional shear strength value has been added for the unoxidized zone of the San Onofre bedrock as encountered during drilling at depth within the hillside. The zone of unoxidized bedrock was observed in limited areas within borings excavated at the site and it has been delineated on the Geotechnical Cross-Sections provided herein, for areas where it has been observed. The material is too hard to sample and has therefore not been specifically tested; it represents the cemented and partially cemented material that can be difficult to excavate, sometimes resulting in drilling refusal with conventional bucket auger drill rigs.

The laboratory testing performed by G.A. Nicoll and Associates, Inc. and others (References), has been gathered and provided in the attached Appendix C, Laboratory Test Results.

TABLE 1

Soil Shear Strength Parameters

Soil Type	ϕ (Degrees)	Cohesion (psf)
Landslide Material, Landslide Rupture Plane, and Silty Clay Bed	19	270
Compacted Fill (Af)	29	200
Weathered San Onofre Breccia (Tso),and Queried San Onofre Breccia	30	500
Unoxidized San Onofre Breccia (Tso), across bedding	39	1,500

3.2 Slope Stability Analyses

Slope stability analyses were based on modeling the two-dimensional geotechnical Cross-Sections A-A' through F-F' for both the Proposed Master Plan and the Alternative. Slope stability analyses for the critical area of the slope at the northeast portion of the site were performed utilizing a conceptual design of caissons (a.k.a. "piers") and tiebacks in order to stabilize the ground supporting the proposed building locations. Caisson depths and tieback array details including unbonded length, strength, and spacing of tiebacks were modeled to increase the static factor of safety to a minimum of 1.5 and pseudo-static factor of safety to a minimum of 1.1. These analyses were performed using the computer program GSTABL7 with STEDwin version 2.002. Block failure modes were analyzed using Janbu's Simplified Method. Pseudo-static analysis was performed utilizing a vertical acceleration coefficient of 0.4g and a horizontal coefficient of 0.15g. The engineering analyses have been provided in Appendix D. The Preliminary Remedial Measures Maps (Sheets 2 and 7) and selected cross-sections depict the proposed tieback and caisson mitigation plan.

The areas depicted by Cross-Sections D-D' and E-E' at the southeast portion of the site have been analyzed for slope stability using the Modified Bishop Method. Factors of safety for the proposed development of the southeast portion of the site were calculated to exceed code minimums. Engineering analyses for Cross-Sections D-D' and E-E' are included in Appendix D.

The proposed new structures to the north of the existing Sanctuary will be protected in their entirety with the caisson and tieback array. The existing Sanctuary structure is founded on bedrock of the San Onofre Formation as reported by Nicoll and additionally determined by LGC Geotechnical based on review of site geologic structure. The Sanctuary building is supported by engineered fill placed on bedrock reviewed and accepted by Nicoll, within a zone where underlying geologic conditions for construction of the Sanctuary are supported by their excavation and analysis of data from Boring BA-1(X) at the outer edge of the structure. In the unlikely event of failure through the engineered fill materials that overlie the projected location of the Silty Clay Bed east of the Sanctuary, a bedrock slope would be left in-place for support of the Sanctuary structure.

For the proposed Master Plan, an additional row of caissons has been recommended south of the tieback system in order to extend the increase in stability gained with the tieback system southward, toward the existing Sanctuary. The caissons are depicted in plan view on the Preliminary Remedial Measures Map (Sheet 2) to the limits of existing engineered fill placed for support of the slope below the Sanctuary. Although presence of caissons in this area would limit potential size of a hypothetical failure east of the Sanctuary, such a failure would require slope repairs to be implemented in accordance with standard geotechnical recommendations.

3.3 Risk Assessment of Unimproved Areas

Slope stability analysis for the slope area to the east of the proposed structures at the northern portion of the site has been performed for estimation of post-construction stability of unimproved areas. The method of averaging the results of slope stability analyses across multiple, equally spaced, parallel cross-sections is an engineering technique for estimating potential for failure in three dimensions. Analysis has been performed for Cross-Sections A-A', B-B', C-C', and two intermediate cross-sections equally spaced between the original three parallel cross-sections. The landslide basal rupture surface has been modeled along with site improvements (tiebacks and caissons) within the five analyses. The average factor of safety against reactivation of the landslide is approximately 1.2. Results of the analyses are presented in Appendix D within the section titled “Risk Assessment of Unimproved Areas”. The line noted as “Approximate Limit of Factor of Safety of 1.5” on the Preliminary Remedial Measures Maps (Sheets 2 and 7) represents the approximate line of demarcation between portions of the site which will possess slope stability factors of safety of at least 1.5 for static and 1.1 for seismic, and portions of the site that do not.

After construction of site improvements in general accordance with the recommendations presented herein, unimproved slope areas will remain at risk for failure. The size of potential failure is significantly reduced, however, and there is some reduction in the risk for global failure as the solution provides for mechanical support of the upper portion of the slope instead of bearing on the lower portion of slope. Practices such as establishing plants, avoiding concentration of water to the subsurface, discouraging rodent activities, and repairing erosion rills that may occur will help to limit potential for failure of unimproved areas. Slope maintenance recommendations will be provided in a future grading plan review report. In the event of failure, slope repairs should be implemented in accordance with geotechnical recommendations on a case-by-case basis.

A typical mudflow or mudslide is a failure of the upper 4 feet of saturated hillside material. The potential for mudslide or mudflow after construction of site improvements is lessened with the implementation of a slope maintenance program within the limits of the property. Potential for mudflow or mudslide for hillside areas outside of the property limits would also be incrementally lessened by the recommended slope maintenance program due to the decreased potential for the upper portion of the slope to fail as a mudflow or mudslide.

It should be noted that the neighboring site to the north was subject to a post-construction landslide during 1991. The Bluffs Development was constructed near the toe of slope area within the Monterey Formation. The Monterey Formation is known for its higher potential for landslide occurrence in comparison to the San Onofre Breccia due to the nature of the material; it is considered weaker than the San Onofre Breccia from a geotechnical perspective. The South Shores Church is sited fully within the San Onofre Breccia, and the proposed tieback and caisson system will tie the development to the stronger material.

3.4 Seismic Design Criteria

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2010 C.B.C. Site coordinates of latitude 33.4880 degrees north and longitude -117.7213 degrees west, which are representative of the site, were utilized in our analyses. The initial results of our analyses for the maximum considered earthquake spectral response accelerations (S_S and S_1) are presented in Table 2A.

TABLE 2A

Seismic Design Values

Selected Parameters from the 2010 C.B.C. Section 1613 - Earthquake Loads	Seismic Design Values
Site Class per Table 1613.5.2	C
Spectral Acceleration for Short Periods (S_S)*	1.629 g
Spectral Accelerations for 1-Second Periods (S_1)*	0.593 g
Site Coefficient F_a per Table 1613.5.3(1)	1.0
Site Coefficient F_v per Table 1613.5.3(2)	1.3

* Calculated from the USGS computer program "Seismic Hazard Curves, Response Parameters and Design Parameters" v5.1.0 (02/10/11)

The spectral response accelerations (S_{MS} and S_{M1}) and design spectral response acceleration parameters (S_{DS} and S_{D1}), adjusted for Site Class C, were evaluated for the site in general accordance with section 1613 of the 2010 C.B.C. These site class adjusted parameters are presented in Table 2B.

TABLE 2B

Seismic Design Values Modified for Site Class C

Selected Parameters from the 2010 C.B.C. Section 1613 - Earthquake Loads	Seismic Design Values Modified for Site Class C
Site Modified Spectral Acceleration for Short Periods (S_{MS}) for Site Class C [Note: $S_{MS} = F_aS_S$]	1.629 g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class C [Note: $S_{M1} = F_vS_1$]	0.771 g
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class C [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.086 g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class C [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.514 g

In accordance with Tables 1613.5.6 (1 & 2), the Seismic Design Category for the subject site is Category D, where $S_{DS} \geq 0.50$g and $S_{D1} \geq 0.20$g.

Section 1803.5.12 of the 2010 C.B.C. states that the PGA for a site may be defined as $S_{DS}/2.5$. The S_{DS} for the subject site has been calculated as 1.086g. Therefore, PGA = 1.086g/2.5 = **0.43g**

4.0 CONCLUSIONS

The following conclusions have been determined to be applicable to the proposed re-development of the subject site.

- The site is feasible for construction and is suitable for the proposed re-development in accordance with both the Proposed Master Plan and Alternative Design from a geotechnical viewpoint, provided the recommendations of this report and a future grading plan review report are implemented.
- The northeast portion of the site will require slope stabilization in order to achieve stable land to the current building code for construction of the Community Life Center Building and the Christian Education Buildings.
- The site is potentially affected by earthquake-induced landslides that can be mitigated by slope stabilization in accordance with the geotechnical recommendations of this report and future reports.
- Seismic design parameters indicate the site is subject to a peak ground acceleration of approximately 0.43g.
- No liquefaction hazard is present, based on our subsurface evaluation and the Seismic Hazard Map applicable to the City of Dana Point.
- Expansive soil potential at the site is anticipated to range from “low” to “moderate”, based on visual observation and testing of on-site, near surface soils in accordance with ASTM D4829 Test Method.
- Groundwater was encountered during the subsurface investigations as random seepages and as a static water table as observed at approximately 90 feet below ground in boring LGC-1.
- It is our opinion that no substantial soil erosion or loss of topsoil (including mudflows and mudslides) in ungraded areas will occur as a result of the proposed development, as long as the recommendations presented here and in future reports are implemented.

5.0 PRELIMINARY RECOMMENDATIONS

The following recommendations are to be considered preliminary, and should be finalized and expanded in a grading plan review report. In addition, all recommendations from LGC Geotechnical should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the City of Dana Point.

Please note that the proposed tieback and caisson solution presented below for mitigation of onsite stabilization issues also significantly lessens the potential for off-site failure of northeastern slope areas in the future. The solution provides for mechanical support of the upper portion of the slope instead of bearing on the lower portion of the slope.

5.1 Mechanical Slope Stabilization

In order to increase the gross stability of the northeast portion of the site to the minimum factor of safety required for new construction, a slope stabilization system consisting of tiebacks and caissons is proposed as presented on the Preliminary Remedial Measures Maps (Sheets 2 and 7). The geologic feature that controls the engineering analysis is labeled Silty Clay Bed on the Geotechnical Maps (Sheets 1 and 6). The feature is angled at depth as shown on the cross-sections. Based on slope stability analysis of the most critical Cross-Section A-A' for the Proposed Master Plan, the proposed tieback and caisson array for stabilization of the area furthest from the design geologic feature is achievable and stabilizes the slope to the required minimum factor of safety of 1.5 for static conditions, and to the minimum factor of safety of 1.1 for pseudo-static conditions. Slope stability analysis is presented in Appendix D.

The tieback array as modeled is recommended to be 5-foot on center for both rows and columns. Recommended preliminary positions of reaction walls, tieback columns, and caissons are presented on the Preliminary Remedial Measures Maps. Tieback columns are shown in cross-sectional view at 5-foot on center vertical spacing showing 4 tiebacks, 3 tiebacks, and 2 tiebacks per column depending on distance to the design feature. Based on the geometry of the design geologic feature (Silty Clay Bed), stabilization of areas closer to the feature requires fewer tiebacks (or lower-capacity tiebacks) and shallower caissons. Stabilization of areas further from the feature requires more, higher-capacity tiebacks and deeper caissons.

The restraining loads needed to stabilize the slope at the location of the highest anticipated loads, Cross-Section A-A' for the Proposed Master Plan, are approximately 360 kips per anchor for the analyzed tieback array, as shown on the slope stability analysis for the cross-section. This load is achievable in accordance with the current standards of tieback installation, using approximately 11 strands per anchor. It is our understanding that loads of up to 420 kips are constructible with standard equipment, using 14-strand anchors. Therefore, there is some room for a greater load in the unlikely event that distance to the design feature was to increase.

There is a great deal of flexibility in the potential design in that an additional row of tieback anchors could be designed to reduce the restraining loads of each anchor, or a row could be removed and the loads increased for areas of lesser distance from the design feature. The maximum load of 360 kips per anchor is an achievable load that will allow excavation of the anticipated access pad geometry for the number of rows proposed at each area for both the Proposed Master Plan and the Alternative Design as represented by Cross-Sections A-A’, B-B’, and C-C’.

Please note that with the Alternative Design, the critical cross-section becomes Cross-Section B-B’; all other tieback wall locations would be pulled back toward the Silty Clay Bed and have lesser loads or fewer tiebacks than the Proposed Master Plan. Restraining loads are approximately 250 kips per anchor at Cross-Section B-B’ in this preliminary design.

Caissons recommended to be constructed in conjunction with the tieback array are modeled to be 3 feet in diameter, and should extend to depths that exceed approximately 40 feet of horizontal setback from the Silty Clay Bed at depth. This relationship is presented on applicable cross-sections for clarity. Grade beams connecting the caissons will be utilized.

For the Proposed Master Plan, additional grade beams will be recommended to tie all caissons supporting the proposed retaining wall east of the Christian Education Buildings to the caissons adjacent to the tieback array, in order to ensure stability. Three locations where the retaining wall is outside of the tieback wall create respective structural triangles in plan view. The caissons supporting the eastern retaining wall will be sufficiently deepened and reinforced to take deflection due to the small wedge of earth between the tieback reaction wall and the retaining wall. Within the structural triangles, interior grade beams and additional caissons may be added by the structural engineer during design. The retaining wall should be constructed on a grade beam supported by the caissons, and designed with geogrid or similar locally stabilizing elements. The caisson array will be tied to the tieback reaction wall within an additionally reinforced grade beam at the base of the tieback wall. A caisson row is recommended to extend past the tiebacks to the south in order to extend the increase in stability gained with the tieback wall toward the existing Sanctuary.

Caissons that are recommended for the horizontal slope setback should be specifically designed in accordance with slope setback/deepened footing requirements as discussed in Section 5.7.

Precise location of the stabilization system relative to structures will be finalized and specific details of the proposed tieback and caisson array and grade beam connections will be designed at the grading plan review phase.

5.2 ***Tieback Access Excavation***

In order to construct the recommended tieback and caisson stabilization system, an excavation will be necessary to achieve access. It is anticipated that the tieback and caisson access excavation will be performed in stages, where the first section is cut down to the level required to install the system, and the next section is cut to the required level while backfilling the first section. Please note that a completed, installed stabilization system does not depend on the presence of backfill for achieving stability, therefore timing of backfill of the access excavation is not critical to the interim stability of the site.

Approximate limits of the proposed tieback access excavation are depicted on the Preliminary Remedial Measures Maps, Sheets 2 and 7.

5.3 Community Life Center and Christian Education Building Retaining Walls

Retaining walls are proposed at the northeast area of the subject site for both the Proposed Master Plan and the Alternative Design. The most structurally significant wall for the Proposed Master Plan is the approximately 270-foot long wall proposed for local support of both the Community Life Center and the walkway and drive aisles adjacent to the Christian Education Buildings. The Alternative Design depicts a similar length of variable retaining walls that are smaller in general and obscured by the Christian Education Buildings in most locations.

For each of the respective designs presented herein, the retaining structure adjacent to the Community Life Center would begin along the north-facing side of the building pad, turn a corner, and extend the length of either the Community Life Building (Master Plan) or the west side of a Christian Education Building (Alternative Plan). Going south, a wall for support of walkways and drive aisles is proposed adjacent to the west side of the Christian Education Building(s). Specifics of these proposed retaining structures have not been provided at this time, however, they are considered feasible for construction from a geotechnical viewpoint. Cross-Sections A-A', B-B', and F-F' generally depict the walls relative to the respective designs. Deepened foundations for the northern boundary of the wall adjacent to the Community Life Center are recommended as presented on the Preliminary Remedial Measures Maps, Sheets 2 and 7, and in profile on the noted cross-sections. See Section 5.7 for further discussion on deepened footings.

For the Proposed Master Plan only, a retaining wall is proposed at the eastern side of the Christian Education buildings that provides for a small area of fill between approximately 6 feet and 12 feet high, supported on caissons. Structural support for the wall is discussed in Section 5.1 titled "Mechanical Slope Stabilization". The retaining wall is depicted on the Preliminary Remedial Measures Map (Sheet 2), and within profiles on Cross-Sections A-A' and C-C'. The additional fill has been modeled on slope stability analyses for the noted cross-sections, as presented in Appendix D.

Once final design plans for the proposed retaining walls are completed, LGC Geotechnical will provide specific geotechnical recommendations for structural design and construction. Provisional geotechnical analysis indicates the proposed retaining walls can be constructed without off-site geotechnical impact.

5.4 Pre-School/Administration Building and Meditation Garden

The Pre-School/Administration Building at the southeastern portion of the site is planned to be contiguous with the adjacent Meditation Garden. For the Alternative Design, the Pre-School/Administration structure is significantly smaller than the Proposed Master Plan and pulled back from the eastern property line. A series of retaining walls have been proposed along the east and south facing outside slope face, to create the curving walls for the Meditation Garden at variable levels, to be combined with water features and landscaping. Cross-Sections D-D' and E-E' for both the Proposed Master Plan and the Alternative Design depict the area in profile, and global slope stability analysis of the cross-sections for each respective design are presented in Appendix D.

Once final design plans for the proposed retaining walls are completed, LGC Geotechnical will provide specific geotechnical recommendations for structural design and construction. Provisional geotechnical analysis indicates the proposed retaining walls can be constructed without off-site geotechnical impact.

5.5 *Existing Crib Wall*

The existing crib wall structure and engineered backfill at the southern boundary of the project was geotechnically reviewed with regards to the additional load of the parking structure to be placed near the top of the crib wall. An exploratory boring was excavated through the approximately thickest portion of engineered fill for confirmation of the competency of the fill placed under observation and testing by Nicoll (1992). Boring LGC-2, depicted on the Geotechnical Maps (Sheets 1 and 6), was sampled, downhole logged, and laboratory testing was performed on representative samples. Boring information and laboratory testing results are presented in Appendix B and C, respectively. Minor tension cracks are visible within the existing parking lot parallel to the top of the ascending slope above the existing crib wall; however, no vertical offset was observed within the relatively old cracks. The approximately 20-year-old certified fill was observed, tested, and determined to be competent for future continued use in support of parking areas. Specific recommendations for construction of new improvements adjacent to the existing crib wall are required in order to ensure no additional structural loads are placed on the wall. Refer to Section 5.7, Deepened Foundations for Top-of-Slope Structures, for additional details.

5.6 *Parking Structure*

A two-story parking structure is proposed within both the Proposed Master Plan and Alternative Design. Within the Alternative Design, however, the majority of the southern boundary of the structure is pulled back from the crib wall by an additional 10 feet in comparison to the Proposed Master Plan. The structure will be constructed with several conventional retaining walls at the northern and western perimeters, and it will overlie a portion of the backfill for the existing crib wall at the southern perimeter. Although actual design loads for the parking structure are not available at this time, we anticipate that all structural loads over existing fill material will be transmitted to bedrock below by caissons or deepened footings in the area of the existing crib wall. Areas of the structure underlain directly by the San Onofre Breccia can be provisionally designed as spread footings.

For evaluation of the parking structure relative to the crib wall, an Existing Crib Wall Exhibit was provided by Adams-Streeter, presented at the rear of text. The exhibit depicts the subsurface configuration of the existing crib wall at approximately the maximum height of the wall, and the relative distance between existing and proposed foundation elements for the parking structure. Cross-Section G-G' by LGC Geotechnical (Sheets 5 and 10) depicts our geotechnical recommendations for construction of the proposed parking structure. The approximate locations of the recommended deepened foundation elements, or caissons, are presented in plan view on the Preliminary Remedial Measures Maps (Sheets 2 and 7). See Section 5.7 for further discussion on deepened footings.

Once final design plans for the parking structure are completed and structural loads are finalized, LGC Geotechnical will provide specific geotechnical recommendations for construction. Provisional geotechnical analysis indicates the structure can be constructed without off-site geotechnical impact.

5.7 Deepened Foundations for Top-of-Slope Structures

The City of Dana Point and the current California Building Code are applicable in determining the appropriate depth of deepened foundations for reducing the required top-of-slope setback for proposed structures. Foundation criteria should be reviewed by LGC Geotechnical based on the final grading plan. Specific foundation systems for each area are not fully designed at this time, however, the following guidelines are recommended.

In general, the intent of the geotechnical slope setback requirements is to ensure the stability of proposed structures. As such, since the majority of the Community Life Center and the Christian Education Buildings are to be founded above an extensive system of slope stabilizing caissons and tiebacks, no additional setbacks are recommended. This condition applies to Geologic Cross-Sections A-A', B-B', and C-C' for both the Proposed Master Plan and the Alternative Design. The Christian Education Buildings are recommended to be founded on conventional footings for both designs. For the Proposed Master Plan, the northwest corner of Christian Education Building No. 2 will require a small zone of deepened footings to ensure the entire foundation is within competent native soils.

The variable height wall at the northern perimeter of the Community Life Center is recommended to be supported by deepened footings in accordance with horizontal setbacks per code. As shown in the slope stability analysis for Cross-Section F-F' that is included within this report (Appendix D), the location does not require global stabilization due to the shallower inclination of the slope, the presence of fill at the toe-of-slope, and slightly more favorable structural geology (apparent dip). However, we recommend that the wall structure at the top of the slope be founded on a deep foundation system to negate the effects of slope creep. The approximate locations of caissons for deepened foundations are presented on the Preliminary Remedial Measures Maps (Sheets 2 and 7). Specific recommendations for these caissons, including anticipated deflection, will be provided in the design phase of the project. The Community Life Center structure is located behind the wall and is recommended to be founded on conventional footings. The entire foundation will be constructed on engineered fill that is a minimum of 5 feet thick.

The Pre-School/Administration Building at the southeastern portion of the site is proposed to be founded on conventional footings. The foundation will be constructed on the engineered fill that is a minimum of 5 feet thick. The retaining walls for the adjacent Meditation Garden will require deepened footings. For geologic Cross-Sections D-D' and E-E', where slopes are relatively gradual below the proposed improvements, we will provide specific foundation setbacks from slope faces at the design phase of the project. As a general rule, we recommend that the base of retaining wall footings be a minimum of 10 feet from slope faces and other habitable structure footings be a minimum of 20 feet from slope faces. These recommendations will be finalized at the grading plan review/design stage of the project.

The southern boundary of the proposed parking structure will require caissons and deepened foundation elements in consideration of its proximity with the existing crib wall near the southern property line, as discussed in the section titled Parking Structure (Section 5.6), and in accordance with the Existing Crib Wall Exhibit (Rear of Text) and Cross-Sections G-G' (Sheets 5 and 10). We anticipate all these caissons will extend through fill to bedrock. Approximate locations of proposed caissons are depicted on the Preliminary Remedial Measures Maps (Sheets 2 and 7).

5.8 Site Earthwork

The proposed remedial grading for the project will include site preparation, design cuts and fills in accordance with the civil engineering plan, overexcavation of structures supported on conventional (non-deepened) footings on cut to fill transitions where the exposed cut is formational material, excavation of an access pad for installation of tiebacks at the eastern boundary of the tieback reaction wall area, and retaining wall and utility line excavation and backfill. Design cuts and fills planned for achieving the terracing effect of the Meditation Garden are intended to work with the natural topography of the area. Both the Proposed Master Plan and Alternative Design incorporate these grading features.

Some export of excess soils is anticipated in order to balance site earthwork. The "South Shores Church Corrective Grading Exhibit, Rough Grade Earthwork Quantities, Sheets C-2.0 through C-2.5" by Adams-Streeter Civil Engineers, Inc. (2013), specifically details the design cuts and fills for the proposed plan. Material that is removed during remedial grading may be placed as fill. Placement and compaction of fill should be performed in accordance with the grading plan review report, local grading ordinances, and under the observation and testing of LGC Geotechnical. General Earthwork and Grading Specifications for Rough Grading have been included as Appendix E for reference. All areas to accept fill placement shall be geotechnically accepted prior to placement of fill.

Design cuts of up to 5 feet and design fills of up to 10 feet are anticipated to be required at the southeast portion of the site, below the proposed Pre-School/Administration structure. The structure is sited within previously placed artificial fill soils and will therefore require minimal remedial grading including surficial reprocessing estimated to be approximately 2 to 3 feet below existing grades in order to moisture condition and re-compact any weathered existing engineered fill. The existing engineered fill placed under observation and testing by Nicoll (1992) was evaluated by LGC Geotechnical within the recently excavated boring LGC-2, and it was found to be generally acceptable for support of future fill and structures constructed in accordance with project specifications. Additionally, a relatively small area of shallow fill at the northern corner of the building will require 5 feet of overexcavation, as depicted in plan view of the Preliminary Remedial Measures Maps, Sheets 2 and 7.

The parking structure is generally proposed to be a variable design cut of up to 10 feet. The parking areas are not recommended to be overexcavated, and the materials that will be exposed at grade are anticipated to be acceptable for construction. Conventional retaining walls, proposed at the parking structure boundaries, will range between approximately 3 and 10 feet in height, and will require standard backcut excavations for construction access. The southern boundary of the parking structure will require additional foundation recommendations as outlined above in Section 5.6, Parking Structure.

The proposed Community Life Center per the Proposed Master Plan is sited over a cut to fill transition of design cut up to 5 feet, and design fill of up to 15 feet for the variable-height retaining wall supporting the overall structure at the northern and eastern boundary. The Alternative Design improves conditions by siting the Community Life Center at a lower elevation, thereby minimizing the amount of fill and height of retaining walls adjacent to that structure. Cross-Sections B-B' (Sheets 3 and 8) depict the proposed geometry of the most critical location in this area for each respective design. To reduce differential settlement, the cut portion of the building footprint is recommended to be overexcavated 5 feet below pad grade. The material will be removed and replaced as engineered fill to achieve pad grade.

The Christian Education Buildings are generally within design cut, up to 18 feet at the west boundary. For the Proposed Master Plan, a very small zone of sliver fill at the northeast corner of the north building of up to 5 feet will be required. Based on the materials observed within the upper portion of Boring LGC-1, it is our opinion that remedial measures were performed prior to placement of engineered fill, and the landslide materials are competent at approximate foundation grade (to be verified during grading). This area will be provided with recommendations for deepened footings as necessary, placing footing foundations into native materials throughout.

The remaining area of important grading activity is the access pad for construction of the proposed tieback reaction wall at the eastern boundary of the Community Life Center and Christian Education Buildings. The approximate elevations and limits of the access pad for each design are depicted on the Preliminary Remedial Measures Maps and detailed in the corrective grading plan by Adams-Streeter. Section 5.2 titled "Tieback Access Excavation" provides additional details regarding the anticipated earthwork for this area. We recommend the access pad be removed in stages and backfilled concurrently, in order to minimize overall disturbance and/or stockpiling activities at the site.

5.9 Geotechnical Role during Construction

During construction of the project, the geotechnical consultant must observe and geologically map native materials within all overexcavation bottoms, design cuts, temporary slopes, and tieback access pad exposures. Areas of pre-existing engineered fill shall be verified to be competent in accordance with project specifications prior to additional fill placement. Landslide materials to be left in place below the Christian Education Buildings shall be verified to be competent for support of structures. Caissons shall be downhole-logged as required in order to verify geologic conditions at regular intervals. More detailed specifications for the geotechnical consultant's role during construction will be provided at the grading plan review phase of work. This will include observation and testing requirements for fill placement, tieback and caisson installation, subsurface drainage, and wall construction.

5.10 Temporary Stability

The most significant temporary slopes that will be exposed during grading of the subject site are the tie-back reaction walls depicted on Cross-Sections A-A', B-B', and C-C' for both the Master Proposed Plan and Alternative Design. The method of construction of the tieback walls is anticipated to be from top to bottom with installation of upper tieback anchors prior to excavation of lower portions of each section of wall. This type of installation will be recommended unless the contractor prefers and defends an alternative that is similarly protective. The individual tieback anchors will provide both temporary and permanent shoring.

The temporary 1:1 (H:V) slopes proposed for interim earthwork construction within the interior of the site are a maximum of 15 feet in height and anticipated to be constructed within bedrock and engineered fill. Temporary slopes are noted on the cross sections herein. These temporary slopes are anticipated to be sufficiently stable for the interim condition. The project geologist should review these slopes during construction and provide additional recommendations in the event that unanticipated geotechnical conditions are observed.

The retaining walls proposed at other locations throughout the subject site are either design fill construction or conventional retaining walls less than 10 feet in height without surcharged backcuts. It is the responsibility of the contractor to construct temporary backcuts for the conventional walls in accordance with OSHA regulations and standard of care for the industry.

Temporary stability of interim slopes and the caisson and tieback stabilization system is not anticipated to be affected by the presence of groundwater at depth within the subject hillside. The groundwater as observed during our recent geotechnical investigation was well below the work area for the tiebacks, at approximately 90 feet below proposed foundation level for new structures. Some minor amounts of groundwater may be present at the bottoms of the deepest proposed caissons; however, the structural design of the caissons will take groundwater into account. The construction method for the deep caissons should include direction of minor amounts of displaced water to approved collection areas as necessary. No mudflow or mudslide due to construction activities is anticipated.

5.11 Subsurface Drainage

Tieback reaction wall backdrains and retaining wall drains should be planned and constructed in accordance with current standards of practice and reviewed by LGC Geotechnical prior to construction. We anticipate the elevation of the lowest tieback reaction wall drainage outlet will allow drainage utilizing the conventional drain system currently proposed for the subject property.

LGC Geotechnical specifically recommends that no purposeful storm water or other infiltration to the subsurface be planned at the site. Review of the Preliminary Water Quality Management Plan and related exhibit (Adam-Streeter, 2012a and 2012b) indicates general conformance with this recommendation. Landscape watering should primarily drain to site surface drainage conveyances. However, as noted in Section 2.6, Infiltration Feasibility, a minimal watering to establish healthy plant growth may be implemented for the Fuel Management areas that generally "mimics ambient rainfall."

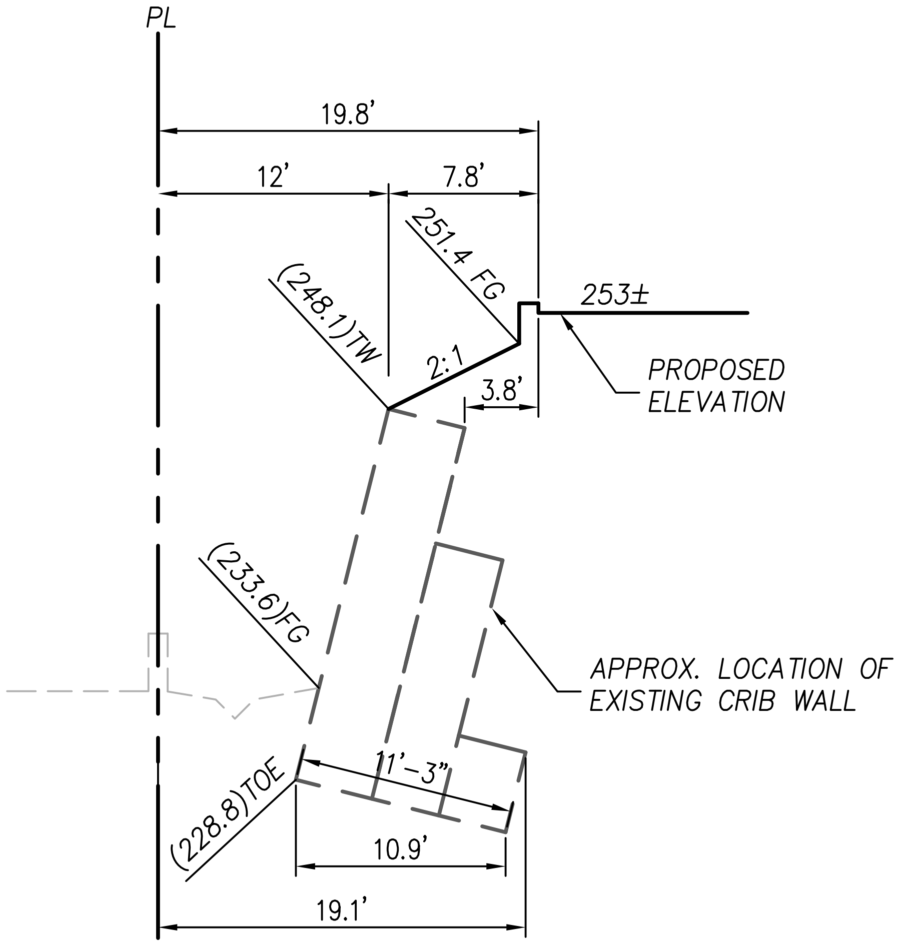
5.12 Grading Plan Review

We have reviewed the referenced preliminary plans (Matlock, 2013 & Adams-Streeter, 2013) and find them to be in general accordance with our geotechnical recommendations. Once the plans are approved, LGC Geotechnical should perform a grading plan review in order to provide full ground stabilization, foundation, and earthwork construction recommendations. Future versions of the development plan and all subsequent plans should be provided to this office for geotechnical review for conformance with the geotechnical recommendations provided in this and subsequent reports.

6.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

It should be understood that LGC Geotechnical has relied on the accuracy of documents, verbal information, and other material and information provided by you and other associated parties in preparation of this report. LGC Geotechnical makes no warranties or guarantees as to the accuracy or completeness of information obtained from or compiled by others.



SECTION A-A

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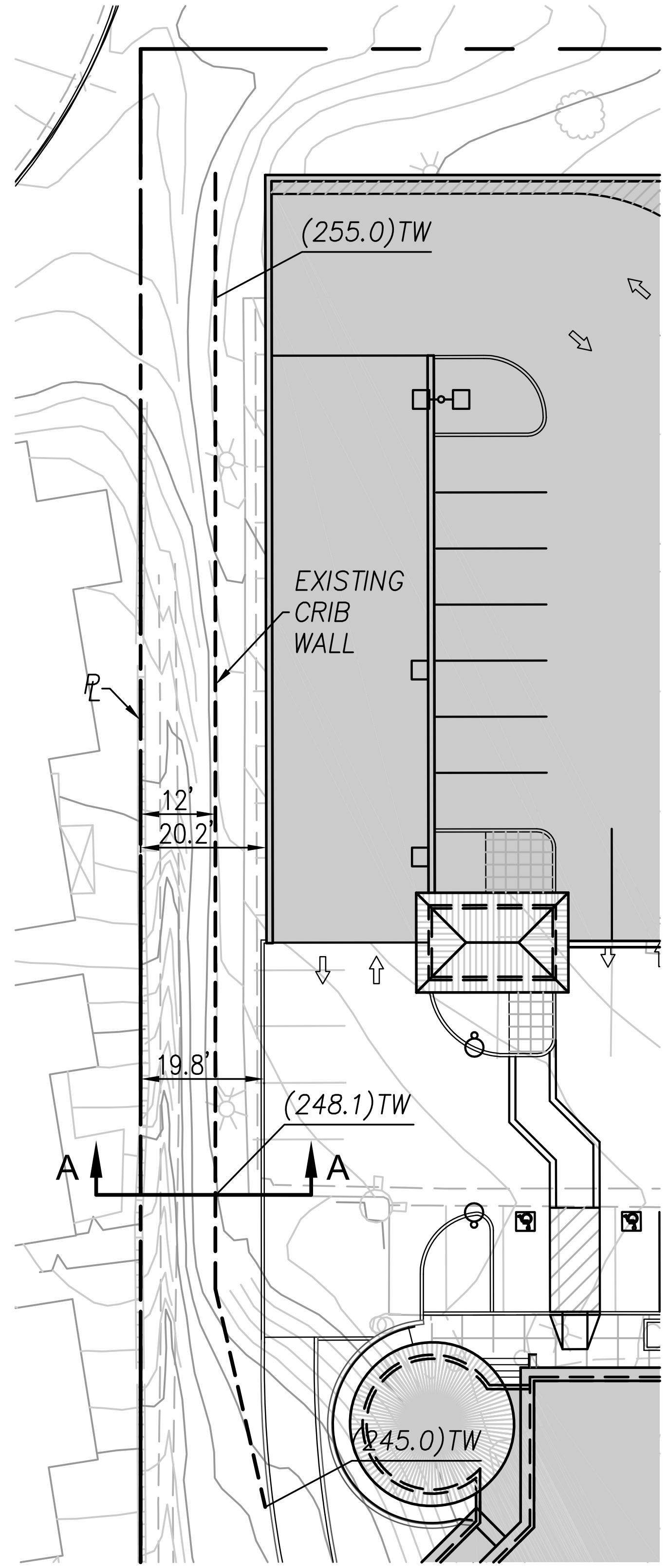

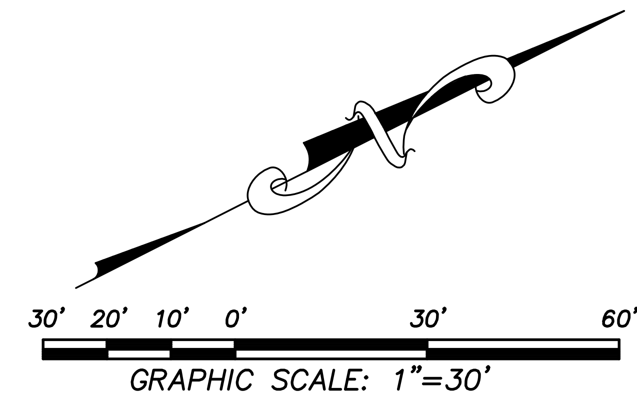

EXISTING CRIB WALL EXHIBIT

SOUTH SHORES CHURCH

PLOT DATE: JUNE 6, 2012

PREPARED BY:

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Appendix A
References

APPENDIX A

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