LGC Geo-Environmental, Inc.
Testing & Inspection

August 24, 2012

Project No. G04711-10

Mr. Will Stout, Forward Planning
THE RANCON GROUP
41391 Kalmia Street, Suite 200
Murrieta, California 92562

Subject: ***Geologic Hazards Evaluation and Updated Preliminary Geotechnical/Fault Investigation, Proposed 9-Acre Medical and Education Center Development and Associated 29.4-Acre Tentative Parcel Map 36492, Located East of Yamas Drive, South of Clinton Keith Road and West of Elizabeth Lane in the City of Wildomar, Riverside County, California***

LGC Geo-Environmental, Inc. (LGC) is pleased to submit herewith our geologic hazards evaluation and updated preliminary geotechnical/fault investigation for the proposed 9-acre medical and education center development and associated 29.4-acre Tentative Parcel Map 36492 located east of Yamas Drive, South of Clinton Keith Road and West of Elizabeth Lane in the City of Wildomar, Riverside County, California.

This report presents the results of our current research of published geologic/geotechnical reports and maps, current geologic mapping and current review of aerial photographs, previous field exploration and previous laboratory testing; as well as our geotechnical and geologic judgment, opinions, conclusions and preliminary recommendations associated with the proposed medical and education center development.

Based on the results of our previous field exploration, current geologic mapping, previous laboratory testing, current geologic and geotechnical engineering evaluations, along with our current review of published literature and the referenced 40-scale plot plan and preliminary grading plan for the site, it is our opinion that the subject site is suitable for the proposed medical and education center development, provided the recommendations presented herein are utilized during the design, grading, and construction. LGC should review final grading plans, as well as any foundation/structural plans when they become available, and revise the recommendations presented herein, if necessary.

It has been a pleasure to be of service to you during the design stages of this project. Should you have any questions regarding the contents of this report or should you require additional information, please do not hesitate to contact us.

31915 Rancho California Road, #200343 Temecula, California 92591•(951) 297-2450 • Fax (951) 344-8214

Respectfully submitted,

LGC Geo-Environmental, Inc.

ROBERT L. GREGOREK II
No. 1257
CERTIFIED
ENGINEERING
GEOLOGIST
Exp. 8/31/13
STATE OF CALIFORNIA

Robert L. Gregorek II, CEG 1257
Geologic Operations Manager

Larry D. Cooley, RCE 54037
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
LARRY DWAIN COOLEY
No. C54037
Exp. 12.31/14
CIVIL
STATE OF CALIFORNIA

RLG/LDC

Distribution: (6) Addressee

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of LGC Geo-Environmental, Inc. (LGC) geologic hazards evaluation and updated preliminary geotechnical/fault investigation for the proposed 9-acre medical and education center development and associated 29.4-acre Tentative Parcel Map 36492 located east of Yamas Drive, South of Clinton Keith Road and West of Elizabeth Lane in the City of Wildomar, Riverside County, California. The purposes of this geologic hazards evaluation and updated preliminary geotechnical/fault investigation was to determine the nature of surface and subsurface soil and bedrock conditions, evaluate their characteristics and provide geotechnical recommendations with respect to grading, construction, foundation design, and other relevant aspects relative to the proposed development of the site. The referenced 80-scale tentative parcel map and the referenced 40-scale plot plan and preliminary grading plan, which was provided, was utilized as the base map for our Geotechnical Map (Plate 1) of the site.

Our scope of services included:

- Review of this firm's previous preliminary geotechnical and geologic reports for the site as well as readily available published topographic, geologic maps, aerial photographs, and pertinent documents regarding the anticipated geologic and geotechnical conditions at the site. (Appendix A).
- Geologic observations and mapping of the existing surface conditions at the site.
- Geotechnical engineering and geologic analysis of the data with respect to the proposed hotel development.
- Analysis of data to address potential geologic issues associated with the proposed development regarding geologic hazards and geotechnical issues.
- Preparation of this report presenting our findings updated conclusions and updated preliminary geotechnical design recommendations for the proposed development.

1.2 Proposed Construction and Grading

The referenced 40-scale plot plan and preliminary grading plan (Plates 2 and 3) prepared by Albert A. Webb and Associates, indicates that the proposed development will consist of 6 structures, associated roadways, parking areas, walk ways, landscape areas and detention basin. Based on information provided by The RANCON Group it is proposed that the structures will be up to one-story to two-story, wood and/or steel frame construction with concrete-floor slabs constructed on-grade. For this type of construction, relatively light to moderate loads will likely be imposed on the underlying soils and bedrock.

Proposed cut and fill depths (exclusive of remedial grading) will be generally be approximately 14 and 8 feet, respectively. Proposed maximum cut and fill slope heights are about 10 feet and 12 feet respectively, at slope ratios of 2:1 (h:v) or flatter. Retaining walls are not proposed at this time.

1.3 Location and Site Description

The subject site is located east of Yamas Drive, South of Clinton Keith Road and West of Elizabeth Lane in the City of Wildomar, Riverside County, California. The general location and configuration of the site is shown on the Site Location Map (Figure 1).

1.4 Topography and Drainage

The topography of the site with elevations varying from approximately 1,385 feet above mean sea level (msl) within the northeastern portion of the site to approximately 1,340 feet above msl within the southeastern portion. Drainage was observed to be generally towards the south and southwest. The southeastern and northwestern corners of the site include existing natural drainage swales that have incised the generally flatter surrounding topography. Drainage was observed to be generally towards the south and southwest.

1.5 Existing Improvements and Vegetation

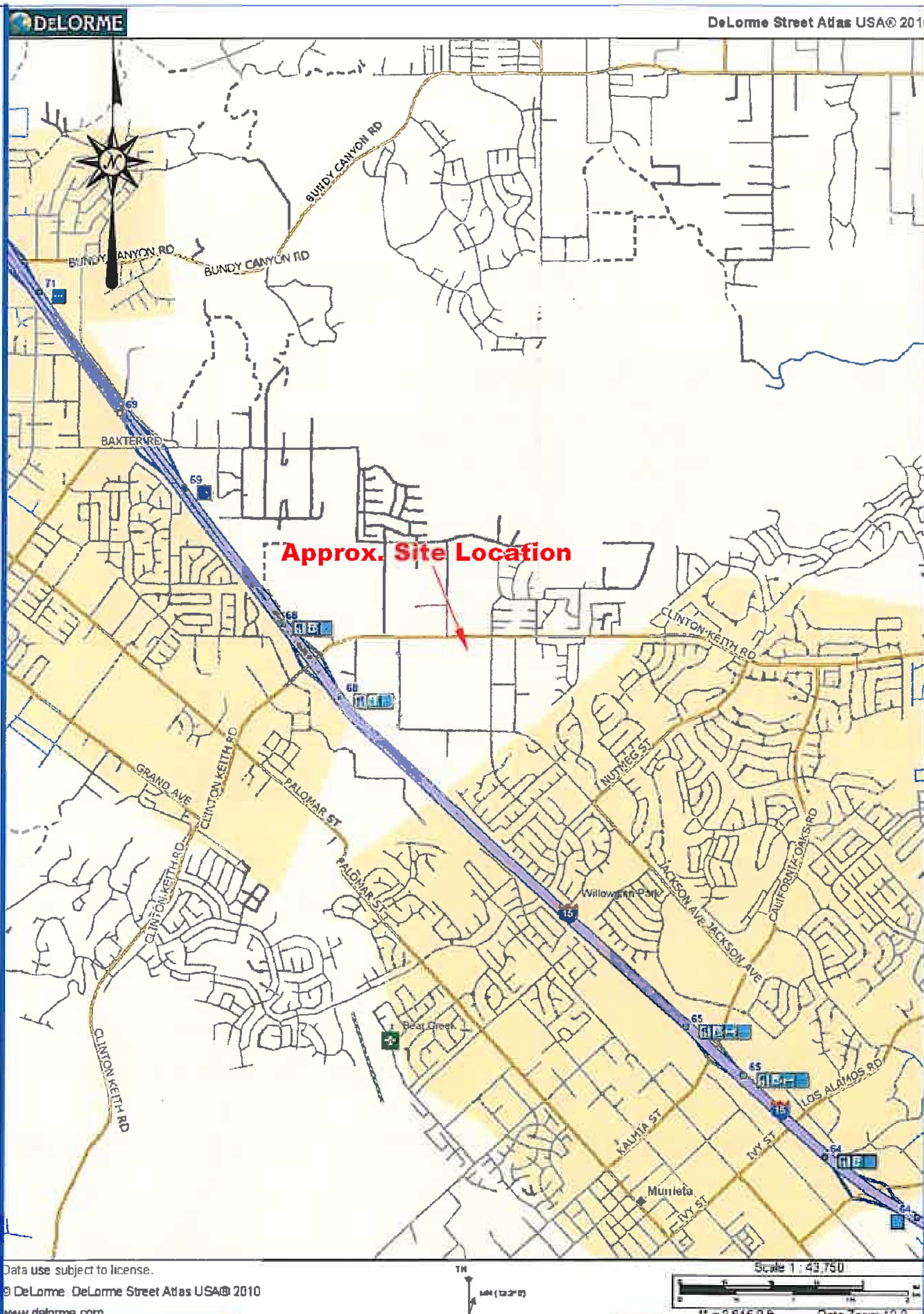
The subject site is currently vacant and has not been developed. However portions of the northeastern property perimeter appear to have been previously graded and improved for adjacent off-site drainage control. Currently a light to moderate cover of annual weeds, grasses and some bushes and trees are present over the site. Scattered concrete, trash and debris were also observed to exist over the surface on portions of the site.

1.6 Research of Previous Geological and Geotechnical Data

This firm researched and reviewed the previous geotechnical and geologic for the site by LGC Inland, Inc. (LGC) as well as available published and unpublished geotechnical reports and geologic data. Pertinent information was incorporated into the conclusions and recommendations presented in our report.

1.7 Aerial Photograph Analysis

Paired stereo aerial photographs for the site and vicinity were reviewed and evaluated by this firm ranging from 1949 through 1997. The photographs were from our files or obtained from Continental Aerial Photo, Inc. Scales of the photographs reviewed (where available) ranged from approximately 1" = 1,667' to approximately 1" = 4,000'. A summary table of the photos reviewed is presented in Appendix A.



LGC

FIGURE 1

SITE LOCATION MAP

Project Name	RANCON MEDICAL CENTER
Project No.	G04-711-10
Geol./ Eng.	RLG/LDC
Scale	AS SHOWN
Date	AUGUST 2012

2.0 FIELD EXPLORATION

2.1 Geologic Mapping

Surface geologic mapping of the site and accessible surrounding areas was accomplished by an engineering geologist from this firm in April and August, 2012, utilizing the referenced 40-scale plot plan and preliminary grading plan conceptual grading plan for plotting geologic units. This information has been plotted on the enclosed 50-scale Geotechnical Map (Plate 1).

2.2 Field Exploration

The previous subsurface exploration on the subject site was performed as part of a preliminary geotechnical investigation of a larger site (LGC, 2005) on March 8, 2005 and involved the excavation of five (5) exploratory borings (B-3 through B-7) to depths ranging from about 11.5 feet to 41.0. feet utilizing a hollow-stem auger drill rig.

In addition excavation of two (2) exploratory fault trenches, each approximately 350 feet long, was performed as part of a fault investigation of a larger site (LGC, 2005) on January 24 through 28, 2005. The fault trenching was performed due to a suspected fault, that may have existed in the southwest corner of the 29.4-acre, near Yamas Drive. A backhoe was utilized to excavate the fault trenches to a maximum depth of 14 feet.

Prior to subsurface work, an underground utilities clearance was obtained from Underground Service Alert of Southern California.

Earth materials encountered within the exploratory borings and fault trenches were classified and logged by a geologist from this firm in accordance with the visual-manual procedures of the Unified Soil Classification System. The approximate locations of the exploratory borings and fault trenches are shown on the Geotechnical Map, Plate 1.

Associated with the exploratory borings was the collection of bulk and relatively undisturbed samples of soil for laboratory testing. Bulk samples consisted of selected soil and bedrock materials obtained at various depth intervals from the exploratory borings. Undisturbed samples were obtained using a 3-inch outside diameter, modified California split-spoon soil sampler lined with brass rings. The soil sampler was driven with successive 30-inch drops of a mechanically driven, 140-pound automatic-trip hammer on the hollow-stem anger drill rig. The central portions of the driven samples were placed in sealed containers and transported to our laboratory for testing. The number of blows required to drive the split-spoon sampler 18 inches for the hollow-stem auger drill rig was recorded in 6-inch increments.

Standard Penetration Tests were also performed in accordance with the American Society for Testing Materials Standard Procedure (ASTM) D1586. This method consisted of driving an unlined standard split-barrel sampler 18 inches into the soil with successive 30-inch drops of the 140-pound automatic trip hammer. Blow counts were recorded for each 6-inch driving increment; however, the number of blows required to drive the split-spoon sampler and the standard split-barrel sampler for the last 12 of the 18 inches was identified as the standard penetration resistance of N-count and recorded in the boring logs. Disturbed soil samples from the unlined standard split-barrel sampler were placed in plastic bags and transported to our laboratory for testing.

2.3 *Laboratory Testing*

During our previous subsurface exploration, representative relatively undisturbed and bulk samples were retained for laboratory testing. Laboratory testing was performed on selected representative samples of onsite soil and/or bedrock materials soil samples and included in-situ dry density and moisture content, maximum dry density and optimum moisture content, Atterberg limits, expansion index, sulfate content, chloride content and R-value tests. A brief description of the laboratory test criteria and test data are presented in Appendix C. In-situ moisture content and dry density are included in the exploratory boring logs (Appendix B).

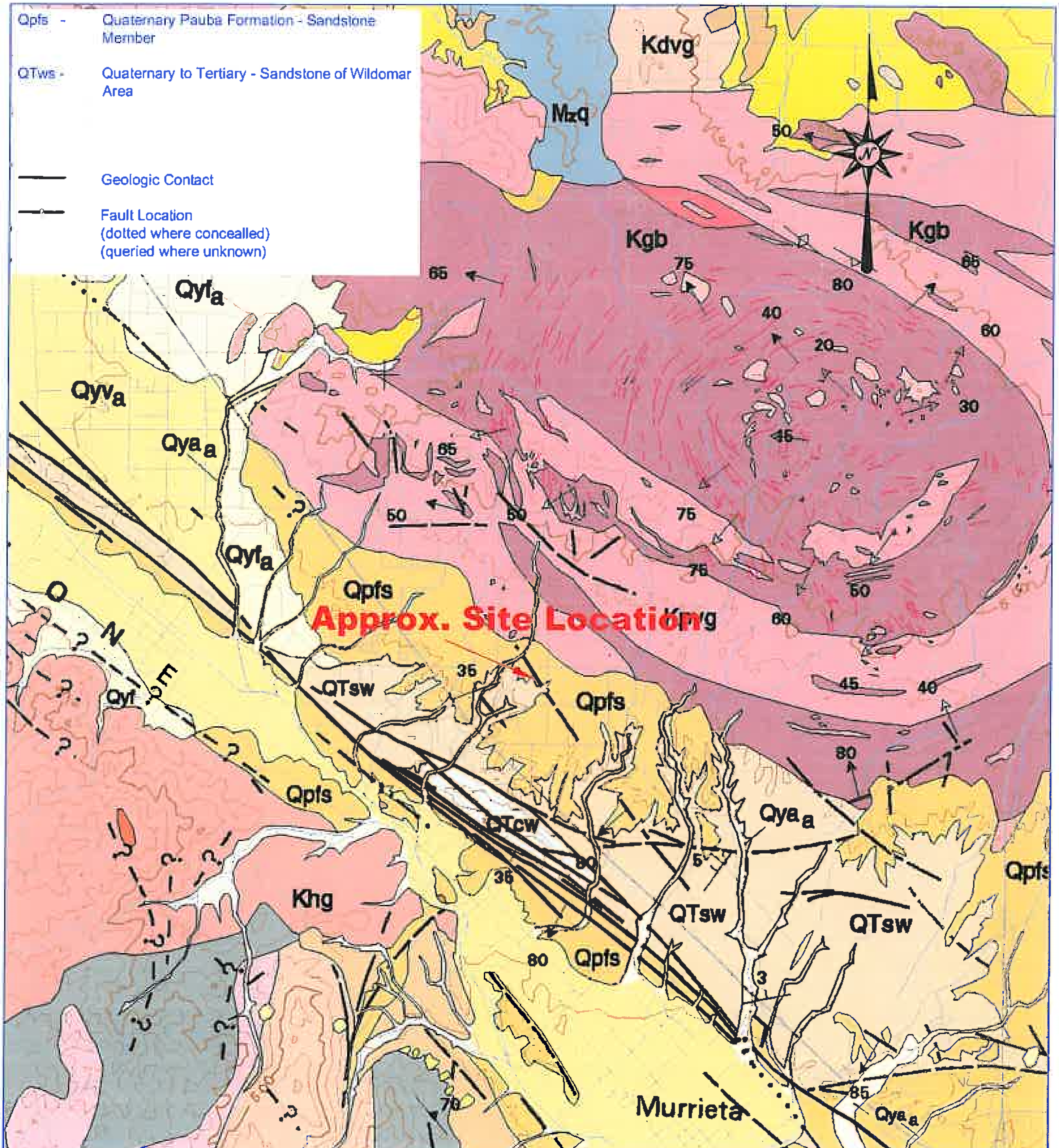
3.0 FINDINGS

3.1 *Regional Geologic Setting*

Regionally, the site is located in the Peninsular Ranges Geomorphic Province of California. The Peninsular Ranges are characterized by steep, elongated valleys that trend west to northwest. The northwest-trending topography is controlled by the Elsinore fault zone, which extends from the San Gabriel River Valley southeasterly to the United States/Mexico border. The Santa Ana Mountains lie along the western side of the Elsinore fault zone, while the Perris Block is located along the eastern side of the fault zone. The mountainous regions are underlain by Pre-Cretaceous, metasedimentary and metavolcanic rocks and Cretaceous plutonic rocks of the Southern California Batholith. Tertiary and Quaternary rocks are generally comprised of non-marine sediments consisting of sandstone, mudstones, conglomerates, and occasional volcanic units. A map of the regional geology is presented on the Regional Geologic Map, Figure 2.

3.2 *Local Geology and Soil/Bedrock Conditions*

Based on our review of available geological and geotechnical literature, current field mapping, and exploratory borings conducted at the site, it is our understanding that the site is primarily underlain by artificial fills (undocumented); topsoil, alluvium and bedrock of the Pauba Formation were encountered during the referenced previous geotechnical investigations. Each unit is described in greater detail below and presented within the exploratory borings logs (Appendix B). The approximate locations of the observed geologic units are depicted on the Geotechnical Map, Plate 1.



Morton, D.M., Hauser, Rachel M., and Ruppert, Kelly R., 2004, Preliminary digital geologic map of the Santa Ana 30' x 60' quadrangle, Southern California, version 2.0: U.S. Geological Survey Open-File Report 99-0172.

LGC

FIGURE 2
REGIONAL GEOLOGIC MAP

Project Name	RANCON MEDICAL DENTER
Project No.	G04711-10
Geol./ Eng.	RLG/LDC
Scale	N.T.S.
Date	AUGUST 2012

- Artificial Fill (map symbol Afu) - Undocumented surficial artificial fill was observed in the northern, southern and eastern portions of the site, primarily as road fills and backfill from previous fault trenches. The fill material, based on our mapping and observations, appears to be comprised of a mixture of sands, silt and clay which were dry to moist and loose to dense, with some gravel and cobbles and possibly pieces of concrete and construction debris. The fills are estimated to be up to approximately 5 feet to 14 feet thick within the areas observed. Based on our observations, the artificial fill is considered non-engineered. Geotechnical reports for the graded road fills along the eastern portion of the site, which may been constructed as part of the Elizabeth Lane improvements of the existing adjacent self storage facility site, were not available at the time of this investigation, which would indicate any overexcavation methods/depths, method of fill placement, fill compaction testing, fill thickness or overall engineering properties. Therefore, further evaluation with additional subsurface exploration and laboratory testing should be performed in areas of proposed construction so appropriated earthwork recommendations, as necessary, may be provided.

- Topsoil (no map symbol) – Topsoil existed over most of the in the site overlying portions of the bedrock. Based on exploratory excavations, mapping and topography, the topsoil is estimated to be about 1.0 foot to 4.5 feet thick and consists generally of medium to dark yellowish brown and brown clayey sand and sandy clay which was damp to very moist, loose to medium dense or soft to stiff and porous.

- Alluvium (map symbol Qal) – Holocene age young alluvial deposits were observed within the northwestern, western, central, eastern and southern portions of the site and consisted generally of sand, silty sand, clayey sand and sandy clay. Based on geologic mapping these materials are estimated to be generally about 3 feet to 10 feet thick and possibly up to 15 feet thick. These materials were generally fine to coarse grained, various shades of brown, gray, yellow and red, damp to wet, and loose to medium dense and soft to firm. These materials are usually porous.

- Bedrock: Pauba Formation (map symbol Qps) – Pleistocene age bedrock of the Pauba Formation exists over the entire site and was encountered in all of the exploratory excavations generally below the artificial fills, topsoil and alluvium. These materials typically consisted of alternating layers of sandstone, silty sandstone, clayey sandstone and sandy claystone, which were generally fine-to-course grained, various shades of brown, tan reds, olive and gray, damp to very moist, moderately hard to very hard, moderately weathered and thickly bedded to massive with some thin sandstone beds. These materials ranged from poorly indurated to well indurated. Portions of the bedrock were observed to be friable. Locally the upper 2 feet to 3 feet of those materials very weathered.

- Bedrock: Cretaceous Tonalite (map symbol Kvt): – In the vicinity of B-3 the Pauba Formation is underlain by granitic rock of the Val Verde Tonalite at depth of about 19 feet. The tonalite was observed to be olive gray, damp to moist and very hard.

3.3 *Landslides*

Review of geologic literature and aerial photographs as well as geologic mapping and previous field exploration does not indicate the presence of landslides on or directly adjacent to the site.

3.4 *Groundwater*

Groundwater was not encountered in the previous borings to the maximum depth explored, approximately 41 feet. Localized water seepage may be present at the alluvium/bedrock contact within the existing drainage courses on the site.

3.5 *Caving*

Caving was not encountered in the exploratory borings or trenches; however localized caving may occur within excavations made into the sandier portions of the on site soil or bedrock.

3.6 *Surface Water*

Based on our review of the referenced 40-scale plot plan and preliminary grading plan, on-site sheet flow is generally trending to the south, east and south. Surface water runoff relative to project design is the purview of the project civil engineer and should be designed to be directed away from the proposed structures and walls.

3.7 *Faulting*

The geologic structure of the Southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. Faults, such as the Newport-Inglewood, Whittier, Elsinore, San Jacinto and San Andreas, are major faults in this system and are known to be active and may produce moderate to strong ground shaking during an earthquake. In addition, the San Andreas, Elsinore and San Jacinto faults are known to have ruptured the ground surface in historic times.

Based on our review of published and unpublished geologic/geotechnical maps and literature pertaining to the site and regional geology, the closest active faults are the Elsinore-Temecula Fault located approximately 3.0 miles from the site and the Elsinore-Glen Ivy Fault located approximately 7.8 miles from the site. Other active faults, within about 20 miles of the subject site, are the Elsinore-Julian Fault, approximately 19.6 miles; and the San Jacinto Fault, approximately 20 miles. These faults are capable of producing a moderate to strong magnitude earthquake

No faults (active, potentially active, or inactive) are known to project through the site, based on review of geologic literature and aerial photographs as well as geologic mapping. The referenced fault investigation (LGC, 2005) also indicated the possible fault in the southwest corner of the site did not exist. The site does not lie within an Alquist-Priolo Earthquake Fault Hazard Zone as defined by the State of California in the Alquist-Priolo Earthquake Fault Hazard Zoning Act.

or a Riverside County Fault Zone. The possibility of damage due to ground rupture is considered negligible since active faults are not known to cross the site.

3.8 ***Seismicity***

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the southern California region, which may affect the site, include soil liquefaction and dynamic settlement. Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that saturated, loose to medium dense, near surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Portions of the alluvial deposits are potentially liquefiable if the become saturated. However alluvial deposits are relatively shallow (about 3 feet to 10 feet) and are underlain by bedrock of the Pauba Formation. Since the alluvial deposits are considered compressible, based on proposed fill loads and anticipated structural loads, these materials should be overexcavated and recompacted in areas of development.

Other secondary seismic effects include shallow ground rupture, and seiches and tsunamis. In general, these secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependant on the distance between the site and causative fault and the onsite geology. A risk assessment of these secondary effects is provided in the following sections.

3.9 ***Settlement Analysis***

The results of our subsurface exploration and laboratory testing indicate the site is underlain by approximately 2 feet to 15 feet of potentially compressible and/or hydro-collapsible soils, consisting of undocumented artificial fill, topsoil alluvium and very weathered bedrock. These materials exhibit the potential to settle or hydro-consolidate under the surcharge of the future proposed fill loads and anticipated structural loads. A portion of the total settlement is due to the potential hydro-consolidation.

4.0 ***CONCLUSIONS AND RECOMMENDATIONS***

4.1 ***General***

Based on the results of our geologic hazards evaluation and updated preliminary geotechnical/fault investigation, it is our opinion that the proposed medical and education center development, as indicated on the referenced 40-scale plot plan and preliminary grading plan, is feasible from a geotechnical and geologic standpoint, provided the following recommendations are incorporated into the design criteria and project specifications. When actual grading plans for the site and foundation/structural plans for the proposed development are available, a comprehensive plan review should be performed by this firm. Depending on the results, additional recommendations may be necessary for geotechnical design parameters for both earthwork and foundations. Grading should be conducted in accordance with local codes, the recommendations within this report and future plan reviews. It is also our opinion that the proposed construction and grading will not adversely impact the geologic stability of adjoining properties.

The following is a summary of the primary geotechnical factors determined from our geotechnical investigation.

- Based on the previous subsurface exploration and review of pertinent geologic/geotechnical maps and reports, the site is underlain by artificial fills (undocumented), topsoil, alluvium and bedrock.
- There are no known landslides impacting the site.
- Groundwater is not considered a constraint for the proposed development.
- The potential for liquefaction is considered remote.
- Active or potentially active faults are not known to exist on the site.
- Previous laboratory test results of the upper soils (artificial fill, topsoil, alluvium and bedrock) indicate a very low expansion potential and negligible potential for soluble sulfate effects on normal concrete.
- The majority of the site is underlain by up to about 2 feet to 15 feet of potentially compressible undocumented artificial fill, topsoil, young alluvium and very weathered bedrock, which may be prone to potential intolerable post-grading settlement and/or hydroconsolidation, under the surcharge of the future proposed structural loads and/or fill loads. These materials should be overexcavated to underlying competent bedrock.
- From a geotechnical perspective, the existing onsite soils appear to be suitable material for use as fill, provided they are relatively free from rocks (larger than 6 inches in maximum dimension), construction debris, and organic material. It is anticipated that the onsite soils may be excavated with conventional heavy-duty construction equipment.

5.0 GEOLOGIC CONSIDERATIONS

5.1 Slopes

No natural slopes or proposed cut slopes with adverse conditions are anticipated.

5.2 Faulting

Geologic hazards due to fault rupture are not known to be present at the subject site.

5.3 Groundwater

Adverse effects on the proposed development resulting from groundwater are not anticipated. A subdrain system should be placed in the bottom of the proposed fill area, following recommended overexcavation, within eastern portion of the site, along Elizabeth Lane where the undocumented fill and alluvium exists. In addition subdrain systems should be placed behind any proposed retaining walls.

5.4 Subsidence

In consideration of the anticipated grading, recommended overexcavations, proposed structures and improvements and subsurface material types and their conditions, unfavorable ground subsidence is not anticipated.

5.5 Landsliding

Landslides or surface failures were not observed at or directly adjacent to the site. As a result, the possibility of the site being affected by landsliding is not anticipated.

5.6 Ground Rupture

Ground rupture due to active faulting is not likely to occur on site due to the absence of known active fault traces. Cracking due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

5.7 Tsunamis and Seiches

Based on the elevation of the proposed development at the site with respect to sea level and its distance from large open bodes of water, the potential of seiche and/or tsunami is considered to be negligible.

6.0 SEISMIC-DESIGN CONSIDERATIONS

6.1 Ground Motions

The site will probably experience ground shaking from moderate to large size earthquakes during the life of the proposed development. Furthermore, it should be recognized that the Southern California region is an area of high seismic risk and that it is not considered feasible to make structures totally resistant to seismic-related hazards.

Structures within the site should be designed and constructed to resist the effects of seismic ground motions as provided in the 2010 CBC Sections 1626 through 1633. The method of design is dependent on the seismic zoning, site characterizations, occupancy category, building configuration, type of structural system and building height.

The following seismic design parameters, presented in Table I, were developed based on the CBC, 2010 and should be used for the proposed structures. A site Coordinate of 33.5944° N, 117.2290° W was used to derive the seismic parameters presented below.

Table 1- Seismic Design Parameters

Seismic Soil Parameters (2010 CBC Section 1613)	
Site Class Definition (Table 1613.5.2)	C
Mapped Spectral Response Acceleration Parameter Ss (for 0.2 second) (Figure 1613.5(3))	1.65
Mapped Spectral Response Acceleration Parameter, S_1 (for 1.0 second) (Figure 1613.5(4))	0.60
Site Coefficient Fa (short period) (Table 1613.5.3(1))	1.0
Site Coefficient F_v (1-second period) (Table 1613.5.2(2))	1.3
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameter S_{MS} (short period) (Eq. 16-37)	1.65
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameter S_{M1} (1-second period) (Eq. 16-38)	0.78
Design Spectral Response Acceleration Parameter, S_{DS} (short period) (Eq. 16-39)	1.10
Design Spectral Response Acceleration Parameter, S_{D1} (1-second period) (Eq. 16-40)	0.52

6.2 Secondary Seismic Hazards

Secondary effects of seismic activity normally considered as possible hazards to a site include several types of ground failure as well as induced flooding. Various general types of ground failures which might occur as a consequence of severe ground shaking of the site include liquefaction, landsliding, ground subsidence, ground lurching and shallow ground rupture. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoils and groundwater conditions, in addition to other factors. Based on the proposed grading and recommended overexcavation of potentially compressible materials within areas of proposed development, indicated on the referenced 40-scale plot plan and preliminary grading plan (Plates 2 and 3), the secondary effects of liquefaction and the other seismic activity noted above are considered unlikely at the site.

Seismically induced flooding which might be considered a potential hazard to a site normally includes flooding due to a tsunami (seismic sea wave), a seiche (i.e., a wave-like oscillation of the surface of water in an enclosed basin that may be initiated by a strong earthquake) or failure of a major reservoir or retention structure upstream of the site. Since the site is located several miles inland from the nearest coastline of the Pacific Ocean at an elevation in excess of approximately 1,360 feet above mean sea level, the potential for seismically induced flooding due to tsunami run-up is considered nonexistent. Since no enclosed bodies of water lie adjacent to the site, the potential for induced flooding at the site due to a seiche is also considered nonexistent.

7.0 GEOTECHNICAL DESIGN PARAMETERS

7.1 Shrinkage/Bulking and Subsidence

Volumetric changes in earth quantities will occur when excavated onsite soils are replaced as properly compacted fill. Following is an estimate of the shrinkage and bulking factors for the various geologic units present onsite. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction that will be achieved during grading.

TABLE 2
Estimated Shrinkage/Bulking

GEOLOGIC UNIT	SHRINKAGE/BULKING PERCENT
Artificial Fill, Undocumented (Afu)	10% to 20% Shrinkage
Topsoil	10% to 15% Shrinkage
Alluvium (Qal)	5% to 15% Shrinkage
Weathered Bedrock	5% to 10% Shrinkage
Bedrock: Pauba Formation (Qps)	0% to 5% Bulking

Subsidence due to recompaction of exposed bedrock, prior to fill placement, and placement of proposed fills, is estimated to be about 0.1 to 0.2 feet.

The above estimates of shrinkage and subsidence are intended as an aid for project engineers in determining earthwork quantities. These are preliminary rough estimates which may vary with depth of removal, stripping losses, field conditions at the time of grading, etc. **However, these estimates should be used with some caution since they are not absolute values.** Contingencies should be made for balancing earthwork quantities based on actual shrinkage and subsidence that occurs during the grading operations.

7.2 Excavation Characteristics

The following excavation characteristics of the various material types at the site have been developed based on LGC's geologic mapping and experience with these materials in the area.

TABLE 3
Excavation Characteristics

GEOLOGIC UNIT	**Easy* Ripping**	**Moderately** Difficult Ripping**	**Oversized Material (>6 inches)**
Artificial Fill, Undocumented (Afu)	X		X
Topsoil	X		
Alluvium (Qal)	X		X
Weathered Bedrock	X		
Bedrock: Pauba Formation (Qps)	X	X	X

* - D-8 with double rippers
** - D-9 with single ripper

Areas of saturated soil may be encountered during overexcavations within the alluvium (Qal) and very weathered bedrock, especially in the lower elevations, within the bottoms of the existing drainage courses.

7.3 *Compressible/Collapsible Soils*

The results of our laboratory testing indicate that the existing undocumented artificial fill, topsoil, alluvium and very weathered bedrock are susceptible to varying degrees of intolerable settlement and/or hydro-consolidation (collapse) when a load is applied or the soil is saturated. Consequently, these materials should be overexcavated to underlying competent bedrock (Qps) and replaced as engineered fill.

8.0 *SITE EARTHWORK*

8.1 *General Earthwork and Grading Specifications*

Earthwork and grading should be performed in accordance with applicable requirements of the grading code of the City of Wildomar and in accordance with the following recommendations prepared by this firm. Grading should also be performed in accordance with the applicable provisions of the attached "Standard Grading Specifications" prepared by LGC (Appendix D), unless specifically revised or amended herein. In case of conflict, the following recommendations shall supersede those included in as part of LGC's General Earthwork and Grading Specifications for Rough Grading (Appendix D).

8.2 *Geotechnical Observations and Testing*

Prior to the start of grading, a meeting should be held at the site with the owner, developer, grading contractor, civil engineer and geotechnical consultant to discuss the work schedule and geotechnical aspects of the grading. Rough grading, which includes clearing, overexcavation, scarification/processing and fill placement, should be accomplished under the full-time observation and testing of the geotechnical consultant. Fills should not be placed without prior approval from the geotechnical consultant.

A representative of the project geotechnical consultant should also be present onsite during grading operations to document proper placement and compaction of fills, as well as to document excavations and compliance with the other recommendations presented herein.

8.3 Clearing and Grubbing

Weeds, grasses, and trees in areas to be graded should be stripped and hauled offsite. Trees to be removed should be grubbed so that their stumps and major-root systems are also removed and the organic materials hauled offsite. During site grading, laborers should clear from fills roots, tree branches and other deleterious materials missed during clearing and grubbing operations.

The project geotechnical consultant or his qualified representative should be notified at the appropriate times to provide observation and testing services during clearing and grubbing operations to observe and document compliance with the above recommendations. In addition, buried structures, unusual or adverse soil conditions encountered that are not described or anticipated, herein should be brought to the immediate attention of the geotechnical consultant.

8.4 Private Sewage System Abandonment

Private sewage systems and/or other subsurface structures that may be encountered should be located, removed and/or properly abandoned. Abandonment and/or removal of septic systems that may exist should be in accordance with local codes. Seepage pits, if abandoned in-place, should be pumped clean, backfilled with gravel or clean sand jetted into place and then capped with 2 feet or more of a 2-sack or more slurry for a distance of 2 feet or more outside the edge of the seepage pit. The top of the slurry cap should be at 10 feet or more below proposed grade.

8.5 Water-Well Capping

Unknown water wells that are encountered within the site which are to be abandoned should be capped and abandoned under permit by the appropriate governmental agency from Riverside County. In addition, a 10-foot or more thick compacted fill blanket, below proposed grade, should be placed above the previously or newly capped water wells.

8.6 Overexcavation and Ground Preparation

The site is underlain by approximately 2 feet to 10 feet, and possibly as much as 15 feet of potentially compressible soils. Therefore existing undocumented artificial fill, topsoil, alluvium and very weathered bedrock are considered unsuitable for support of proposed fills, structures, and/or improvements, and should be overexcavated to expose underlying competent bedrock or existing compacted engineered artificial fill. Where overexcavation and grading do not provide 5 feet or more of fill below finished grade within areas for proposed structures or walls, the area should be overexcavated to 5 feet or more below proposed grade or 3 feet or more below bottoms of footings or walls, whichever is deeper. Actual depths of overexcavation should be evaluated upon review of final grading and foundation plans as well as during grading on the basis of observations and testing during grading by the project geotechnical consultant.

Water intrusion and/or saturated subgrade soils is expected within the alluvium (Qal) and very weathered bedrock during overexcavation, especially in the lower elevations, within the existing drainage courses.

Prior to placing engineered fill, exposed bottom surfaces in each overexcavated area should first be scarified to a depth of approximately 6 inches, watered or air-dried as necessary to achieve a uniform moisture content of optimum or higher and then compacted in place to a relative compaction of 90 percent or more (based on American Standard of Testing and Materials [ASTM] Test Method D1557).

The estimated locations, extent and approximate depths for overexcavation of unsuitable materials, within the areas of the proposed 9-acre medical and education center development portion of the site, are indicated on the enclosed Geotechnical Map, Plate 1. The geotechnical consultant should be provided with appropriate survey staking during grading to document that depths and/or locations of recommended overexcavation are adequate.

Sidewalls for overexcavations greater than 5 feet in height should be no steeper than 1:1 (h:v) and should be periodically slope-boarded during their excavation to remove loose surficial debris and facilitate mapping. Flatter excavations may be necessary for stability.

The grading contractor will need to consider appropriate measures necessary to excavate adjacent existing improvements adjacent to the site without endangering them due to caving or sloughing.

8.7 Fill Suitability

Soil materials excavated during grading are generally considered suitable for use as compacted fill provided they do not contain significant amounts of trash, vegetation, construction debris and oversize material.

8.8 Subdrains

Following overexcavation of the alluvium and weathered bedrock, in the lower elevations within the eastern portion of the site along Elizabeth Lane, a subdrain should be installed where the ultimate depth of fill below proposed grade exceeds approximately 10 feet. Actual locations should be determined by the geotechnical consultant once conditions are exposed during grading. The subdrain will help mitigate potential buildup of hydrostatic pressures below compacted fill due to infiltration of sub- surface and surface waters. It should be noted that while performing overexcavation, groundwater may be encountered near the bedrock contact that may require temporary diversion of the water during installation of the subdrain, and during placement of compacted fill. The approximate tentative location of the recommended subdrain is indicated on the enclosed geotechnical map Plate 1. Typical details for subdrains are shown in the attached LGC "Standard Grading Specifications" (Appendix D).

8.9 ***Oversized Material***

Oversized material that may be encountered during grading, greater than 6 inches, should be reduced in size or removed from the site

8.10 ***Cut/Fill Transitions and Differential Fill Thicknesses***

To mitigate distress to structures and walls, related to the detrimental effect of differential settlement, the cut portions should be eliminated from cut/fill transition areas in order that the entire structure or wall is founded on a uniform bearing material. This should be accomplished by overexcavating the "cut" portions and shallow fill portions 5 feet or more below proposed pad grade or 2 feet below proposed footings, whichever is deeper and replacing the excavated materials as properly compacted fill. Recommended depths of overexcavation are provided in the following table:

[illegible] LL ("fill" portion)	DEPTH O[illegible]
Up to 10 feet	5 feet
Greater than 10 feet	One-third the maximum thickness of fill placed on the "fill" portion (15 feet maximum)

8.11 ***Benching***

Where compacted fills are to be placed on natural slope surfaces inclining at 5:1 (h:v) or greater, the ground should be excavated to create a series of level benches, which are at least a minimum height of 4 feet, excavated into competent materials. Typical benching details are shown in the attached LGC "Standard Grading Specifications" (Appendix D).

8.12 ***Fill Placement***

Fills should be placed in lifts no greater than 8 inches in uncompacted thickness, watered or air-dried as necessary to achieve a uniform moisture content of at least optimum moisture content and then compacted in place to relative compaction of 90 percent or more. Fills should be maintained in a relatively level condition. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with ASTM Test Method D1557.

8.13 Inclement Weather

Inclement weather may cause rapid erosion during mass grading and/or construction. Proper erosion and drainage control measures should be taken during periods of inclement weather in accordance with Riverside County and California State requirements.

9.0 SLOPE CONSTRUCTION

9.1 Slope Stability

Based on the referenced 40-scale plot plan and preliminary grading plan both cut and fill slopes are proposed. at slope ratios of 2:1 (h:v) or flatter and should be grossly and surficially stable.

9.2 Temporary Excavations

Temporary excavations varying up to a height of approximately 5 feet to 15 feet below existing grades will be necessary to accommodate the recommended overexcavation of the unsuitable soil or bedrock materials. Based on the physical properties of the onsite soils, temporary excavations exceeding 5 feet in height should be cut back at a ratio of 1:1 (h:v) or flatter, for the duration of the overexcavation and recompaction of unsuitable soil material. Temporary slopes excavated at the above slope configurations are expected to remain stable during grading operations. However, the temporary excavations should be observed by a representative of the project geotechnical consultant for any evidence of potential instability. Depending on the results of these observations, revised slope configurations may be necessary.

Other factors which should be considered with respect to the stability of the temporary slopes include construction traffic and storage of materials on or near the tops of the slopes, construction scheduling, presence of nearby walls or structures on adjacent properties, and weather conditions at the time of construction. Applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should also be followed.

10.0 POST-GRADING CONSIDERATIONS

10.1 Control of Surface Water and Drainage Control

Positive-drainage devices, such as sloping sidewalks, graded-swales and/or area drains, should be provided to collect and direct water away from the structure and slopes. Neither rain nor excess irrigation water should be allowed to collect or pond against building foundations. Roof gutters and downspouts should be provided on the sides of structures. Drainage should be directed to adjacent driveways, adjacent streets or storm-drain faculties. The ground surface adjacent to the structures should be sloped at a at a gradient of at least 5 percent for a distance of at least 10 feet, and further maintained by a swale or drainage path at a gradient of at least 2 percent. Where necessary, drainage paths may be shortened by use of area drains and collector pipes.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made. Over watering must be avoided.

10.2 *Utility Trenches*

Utility-trench backfill within roadways, utility easements, under walls, sidewalks, driveways, floor slabs and any other structures or improvements should be compacted. The onsite soils should generally be suitable as trench backfill provided they are screened of rocks and other material over 3 inches in diameter and organic matter. Trench backfill should be compacted in uniform lifts (generally not exceeding 6 inches to 8 inches in uncompacted thickness) by mechanical means to at least 90 percent relative density (per ASTM Test Method D1557).

Where onsite soils are utilized as backfill, mechanical compaction should be used. Density testing, along with probing, should be performed by the project geotechnical consultant or his representative, to document proper compaction.

If trenches are shallow and the use of conventional equipment may result in damage to the utilities; clean sand, having sand equivalent (SE) of 30 or greater, should be used to bed and shade the utilities. Sand backfill should be densified. The densification may be accomplished by jetting or flooding and then tamping to ensure adequate compaction. A representative from LGC should observe, probe, and test the backfill to verify compliance with the project specifications.

Utility-trench sidewalls deeper than 5 feet should be laid back at a ratio of 1:1 (h:v) or flatter or braced. A trench box may be used in lieu of shoring. If shoring is anticipated, LGC should be contacted to provide design parameters.

To avoid point-loads and subsequent distress to clay, cement or plastic pipe, imported sand bedding should be placed 1 foot or more above pipe in areas where excavated trench materials contain significant cobbles. Sand-bedding materials should be compacted and tested prior to placement of backfill.

Where utility trenches are proposed parallel to building footings (interior and/or exterior trenches), the bottom of the trench should not be located within a 1:1 (h:v) plane projected downward from the outside bottom edge of the adjacent footing.

11.0 Preliminary Foundation Design Recommendations

11.1 General

Provided site grading is performed in accordance with the recommendations of this report, conventional shallow foundations are considered feasible for support of the proposed hotel structures. Tentative foundation recommendations are provided herein. However, these recommendations may require modification depending on as-graded conditions existing within the building sites upon completion of grading.

11.2 Allowable-Bearing Values

An allowable-bearing value of 1,500 pounds per square foot (psf) may be used for 24-inch square pad footings and 12-inch or more wide continuous footings founded completely within in compacted fill or competent bedrock at a depth of 12 inches or more below the lowest adjacent final grade. This value may be increased by 20 percent for each additional foot of width and depth, to a value no greater than 2,500 psf. The recommended allowable-bearing value includes both dead and live loads and may be increased by one-third for short-duration wind and seismic forces.

11.3 Settlement

Based on the general settlement characteristics of the compacted fill and in-situ bedrock, as well as the aforementioned overexcavation recommendations and anticipated loading, it is estimated that the total settlement of conventional footings will be approximately 0.75 inch. Differential settlement is expected to be 0.50-inch over 30 feet. It is anticipated that the majority of the settlement will occur during construction or shortly thereafter as building loads are applied.

The above settlement estimates are based on the assumption that the grading will be performed in accordance with the grading recommendations presented in this report and that the project geotechnical consultant will observe or test the soil conditions in the footing excavations.

11.4 Lateral Resistance

A passive earth pressure of 250 psf per foot of depth to a value up to 250 psf may be used to determine lateral-bearing resistance for footings. The above values may be increased by one-third when designing for short-duration wind or seismic forces. In addition, a coefficient of friction of 0.35 times the dead-load forces may be used between concrete and the supporting soils to determine lateral-sliding resistance.

The above values are based on footings placed directly against compacted fill or undisturbed native soil. In the case where footing sides are formed, backfill placed against the footings should be compacted to 90 percent or more of maximum dry density as determined by ASTM D1557.

11.5 Footing Setbacks From Descending Slopes

Where structures are proposed near the tops of descending graded or natural slopes, the footing setbacks from the slope face should conform with the 2010 CBC Figure 1808.7.1. The required setback is H/3 (one-third the slope height) measured along a horizontal line projected from the lower outside face of the footing to the slope face. The footing setbacks should be 5 feet or more where the slope height is 15 feet or less and vary up to 40 feet where the slope height exceeds 15 feet.

11.6 Building Clearances From Ascending Slopes

Building setbacks from ascending graded or natural slopes should conform with the 2010 CBC Figure 1808.7.1 that requires a building clearance of H/2 (one-half the slope height) varying from 5 to 15 feet. The building clearance is measured along a horizontal line projected from the toe of the slope to the face of the building. A retaining wall may be constructed at the base of the slope to achieve the required building clearance.

11.7 Footing Observations

Footing excavations should be observed by the project geotechnical consultant to document that they have been excavated into competent bearing soils. The foundation excavations should be observed prior to the placement of forms, reinforcement or concrete. The excavations should be trimmed neat, level and square. Loose, sloughed or moisture-softened soil should be removed prior to concrete placement.

Excavated materials from footing excavations should not be placed in slab-on-ground areas unless the soils are compacted to 90 percent or more of maximum dry density as determined by ASTM D1557.

11.8 Expansive Soil Considerations

Results of previous preliminary laboratory tests by LGC indicate onsite soil and bedrock materials exhibit expansion potentials ranging from **VERY LOW** to **LOW** in accordance with 2010 CBC, Chapter 18. However, expansive soil conditions should be evaluated for individual building pads during and at the completion of rough grading to observe and document the anticipated conditions. The design and construction details presented herein are intended to provide recommendations for the levels of expansion potential which may be evident at the completion of rough grading. Furthermore, it should be noted that additional slab thickness, footing sizes and/or reinforcement more stringent than the recommendations that follow should be provided as recommended by the project architect or structural engineer.

11.9 Footing/Floor Slabs – Very Low Expansion Potential

The following are our recommendations where foundation soils exhibit **VERY LOW** expansion potential as classified in accordance with 2010 CBC. For this condition, it is recommended that footings and floors be constructed and reinforced in accordance with the following criteria. However, additional slab thickness, footing sizes and/or reinforcement may be required by the project architect or structural engineer.

- ***Footings***

  - Exterior continuous footings should be founded into compacted engineered fill below the lowest adjacent final grade at minimum depths of 18 inches deep for one-story to two-story. Interior continuous footings may be founded at a depth of 12 inches or greater into compacted engineered fill below the lowest adjacent final grade. Continuous footings should have a minimum width of 15 inches or more for one-story and two-story structures.

  - Continuous footings should be reinforced with four (4) No. 4 bars, two top and two bottom.

  - Interior isolated pad footings should be 24 inches or more square and founded at a depth of 12 inches or more below the lowest adjacent grade. Footings should be reinforced in accordance with the structural engineer's recommendation.

  - Exterior pad footings should be 24 inches or more square and founded at a depth of 18 inches or more below the lowest adjacent grade. Footings should be reinforced in accordance with the structural engineer's recommendations.

- ***Floor Slabs***

  - Concrete floor slabs should be 6 inches or more thick and reinforced with No. 3 bars spaced 24 inches or less on-centers, both ways. Slab reinforcement should be supported on concrete chairs or bricks so that the desired placement is near mid-depth.

  - Concrete floors should be underlain with a moisture-vapor retarder consisting of 15-mil thick vapor barrier. Laps within the membrane should be sealed and overlapped 12 inches. Two inches or more of clean sand should be placed above and below the membrane to promote uniform curing of the concrete.

  - Prior to placing concrete, subgrade soils should be thoroughly moistened to approximately 100% of optimum moisture content to promote uniform curing of the concrete and reduce the development of shrinkage cracks.

  -

11.10 Footing/Floor Slabs – Low Expansion Potential

The following are our recommendations where the foundation soils exhibit a **LOW** expansion potential as classified in accordance with the 2010 CBC, Chapter 18. The 2010 CBC Section 1808.6 specifies that slab-on-ground foundations resting on soils with an expansion index greater than 20 require special design considerations in accordance with the 2010 CBC, Chapter 18, or by soil stabilization by geotechnical recommendations as approved by the building official. We recommend using an assumed effective plasticity index of 12.

The design and construction recommendations that follow may be considered for reducing the effects of **LOW** expansion soils. These recommendations have been based on the previous experience of LGC on projects with similar soil conditions rather than the design criteria defined in the 2010 CBC, Chapter 18. Although construction performed in accordance with these recommendations has been found to reduce post-construction movement and/or cracking, they generally do not mitigate potential effects of expansive soil action. The owner, architect, design civil engineer, structural engineer and contractors must be made aware of the expansive soil conditions which exist at the site. However, additional slab thickness, footing sizes and/or reinforcement may be required by the project architect or structural engineer

- ***Footings***

  - Exterior continuous footings should be founded into compacted engineered fill below the lowest adjacent final grade at minimum depths of 18 inches deep for one-story to two-story construction. Interior continuous footings may be founded at a depth of 12 inches or greater into compacted engineered fill below the lowest adjacent final grade. Continuous footings should have a minimum width of 15 inches or more for one-story and two-story structures. Continuous footings should be reinforced with four (4) No. 4 bars, two top and two bottom.

  - Interior isolated pad footings should be 24 inches or more square and founded at a depth of 12 inches or more below the lowest adjacent grade. Footings should be reinforced in accordance with the structural engineer's recommendation.

  - Exterior pad footings should be 24 inches or more square and founded at a depth of 18 inches or more below the lowest adjacent grade. Footings should be reinforced in accordance with the structural engineer's recommendations.

- ***Floor Slabs***

  - Unless a more stringent design is recommended by the architect or the structural engineer, we recommend a slab thickness of 6 inches or greater with No. 3 bars spaced 18 inches or less on-centers, both ways. Slab reinforcement should be supported on concrete chairs or bricks so that the desired location near mid-height is achieved.

- Concrete floors should be underlain with a moisture-vapor retarder consisting of 15-mil thick vapor barrier. Laps within the membrane should be sealed and overlapped 12 inches. Two inches or more of clean sand should be placed above and below the membrane to promote uniform curing of the concrete.

- Prior to placing concrete, subgrade soils should be thoroughly moistened to approximately 110% of optimum moisture content to promote uniform curing of the concrete and reduce the development of shrinkage cracks.

12.0 RETAINING WALLS

12.1 Lateral Earth Pressures and Retaining Wall Design Parameters

Conventional foundations for retaining walls within properly compacted fill (resting on competent bedrock) or within competent bedrock should be embedded at least 24 inches below lowest adjacent grade. At this depth, an allowable bearing capacity of 1,500 psf may be assumed for retaining walls founded in competent compacted fill or bedrock of the Pauba Formation.

The following are lateral earth pressures are recommended for the proposed retaining walls. The recommended lateral pressures for approved on-site soils (with an expansion index of 20 or less) for level or sloping backfill are presented on Table 4. Onsite soil should be screened of rocks and other material over 3 inches in diameter.

TABLE 4

Lateral Earth Pressures

CONDITIONS	*EQUIVALENT FLUID WEIGHT (pcf)*	
	Level Backfill	*2:1 Backfill Sloping Upwards*
Active	55	65
At-Rest	65	95
Passive	250	—

For sliding resistance, the friction coefficient of 0.35 may be used at the concrete and soil interface. Wall footings should be designed in accordance with structural considerations. The passive resistance value may be increased by one-third when considering loads of short duration such as wind or seismic loads.

Embedded structural walls should be designed for lateral earth pressures exerted on them. The magnitude of these pressures depends on the amount of deformation that the wall can yield under load. If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at-rest" conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the "passive" resistance.

The equivalent fluid pressure values assume free-draining conditions. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

12.2 Footing Embedments

The base of retaining wall footings constructed on level ground may be founded at a depth of 24 inches or more below the lowest adjacent final grade. Where retaining walls are proposed on or within 15 feet from the top of an adjacent descending fill slopes, the footings should be deepened such that a horizontal clearance of H/3 or more (one-third the slope height) is maintained between the outside bottom edges of the footings and the face of the slope but not to exceed 15 feet nor be less than 5 feet. The above recommended footing setbacks are preliminary and may be revised based on site specific soil conditions. Footing or pier excavations should be observed by the project geotechnical representative to document that the footing trenches have been excavated into competent bearing soils and to the embedments recommended above. These observations should be performed prior to placing forms or reinforcing steel.

12.3 Drainage

Surcharge loading effects from the adjacent structures should be evaluated by the geotechnical and structural engineers. All retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. The outlet pipe should be sloped to drain to a suitable outlet. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. If such seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential.

Weep holes or open vertical masonry joints should be provided in retaining walls 3 feet or less in height to reduce the likelihood of entrapment of water in the backfill. Weep holes, if used, should be 3 inches or more in diameter and provided at intervals of 6 feet or less along the wall. Open vertical masonry joints, if used, should be provided at 32-inch or less intervals. A continuous gravel fill, 12 inches by 12 inches, should be placed behind the weep holes or open masonry joints. The gravel should be wrapped in filter fabric to reduce infiltration of fines and subsequent clogging of the gravel. Filter fabric may consist of Mirafi 140N or equivalent.

In lieu of weep holes or open joints, for retaining walls less than 3 feet, a perforated pipe and gravel subdrain may be used. Perforated pipe should consist of 4-inch or more diameter PVC Schedule 40 or ABS SDR-35, with the perforations laid down. The pipe should be embedded in 1.5 cubic feet per foot of 0.75 or 1.5-inch open graded gravel wrapped in filter fabric. Filter fabric may consist of Mirafi 140N equivalent.

Retaining walls greater than 3 feet high should be provided with a continuous backdrain for the full height of the wall. This drain could consist of geosynthetic drainage composite, such as Miradrain 6000 or equivalent, or a permeable drain material, placed against the entire backside of the wall. If a permeable drain material is used, the backdrain should be 1 or more feet thick. Caltrans Class II permeable material or open graded gravel or crushed stone (described above) may be used as permeable drain material. If gravel or crushed stone is used, it should have less than 5 percent material passing the No. 200 sieve. The drain should be separated from the backfill with a geofabric. The upper 1 foot of the backdrain should be covered with compacted fill. A drainage pipe consisting of 4-inch diameter perforated pipe (described above) surrounded Figure 3 Retaining Wall Detail by 1 cubic foot per foot of gravel or crushed rock wrapped in a filter fabric should be provided along the back of the wall. The pipe should be placed with perforations down, sloped at 2 percent or more and discharge to an appropriate outlet through a solid pipe. The pipe should outlet away from structures and slopes. The outside portions of retaining walls supporting backfill should be coated with an approved waterproofing compound to inhibit infiltration of moisture through the walls.

12.4 Temporary Excavations

The retaining walls should be constructed and backfilled as soon as possible after backcut excavations are constructed. Prolonged exposure of backcut slopes may result in some localized slope instability. To facilitate retaining wall construction, the lower 5 feet of temporary slopes may be cut vertical and the upper portions exceeding a height of 5 feet should be cut back at a gradient of 1:1 (h:v) or flatter for the duration of construction. However, temporary slopes should be observed by the project geotechnical consultant for evidence of potential instability. Depending on the results of these observations, flatter slopes may be necessary. The potential effects of various parameters such as weather, heavy equipment travel, storage near the tops of the temporary excavations and construction scheduling should also be considered in the stability of temporary slopes. Water should not be permitted to drain away from the slope. Surcharges, due to equipment, spoil piles, etc., should not be allowed within 10 feet of the top of the slope.

All excavations should be made in accordance with Cal/OSHA. Excavation safety is the sole responsibility of the contractor.

12.5 Retaining Wall Backfill

The retaining wall backfill soils (with an expansion index of 20 or less) should be placed in 6 to 8 inch loose lifts, watered or air-dried as necessary to achieve near optimum moisture conditions and compacted to at least 90 percent relative density (based on ASTM Test Methods D2922 and D3017).

13.0 PRELIMINARY PAVEMENT DESIGN

Structural pavement section design recommendations presented herein are based on a soil samples recovered during our referenced previous geotechnical investigation (2004).

However, it should be understood that the soil material exposed during grading may differ from the materials sampled and tested during this investigation. Therefore, preliminary pavement recommendations are subject to verification and possible revision based on any revised traffic indices as well as sampling and testing of subgrade soils that exist after rough grading.

For planning and design purposes, we have prepared the following preliminary pavement sections based on an R-value testing results on a near surface soil sample collected. R-value testing, from LGC's referenced geotechnical report (2005), indicated an R-value of 13. Based on the **assumed Traffic Indices** (T.I.'s), of 5.0, 6.5 and 8.0., Table 5 presents recommended preliminary pavement designs for a range of assumed traffic conditions.

TABLE 5
Preliminary Pavement Design

AREA	ASSUMED TRAFFIC INDEX	DESIGN R-VALUE	ASPHALTIC CONCRETE (AC) (feet)	AGGREGA[redacted] BASE (AB[redacted] (feet)
Parking Lot and Auto Drive Areas	5.0	13	0.35	0.50
Entrance Aprons and Heavy Traffic Areas	6.5	13	0.45	0.85
Exterior Streets or Roadway Areas	8.0	13	0.50	1.20

Subgrade soil immediately below the aggregate base (base) should be compacted to a minimum of 95 percent relative compaction based on ASTM Test Method D1557 to a minimum depth of 12 inches. Final subgrade compaction should be performed prior to placing base or asphaltic concrete and after all utility trench backfills have been compacted and tested.

Base materials should consist of crushed aggregate base conforming to Section 200-2 of Greenbook. The upper 12 inches of the subgrade soils and all aggregate base materials should be compacted to at least 95 percent of the laboratory maximum dry density determined in accordance with ASTM D1557.

Our preliminary pavement recommendations should be considered as an assumed minimum and can be revised once actual Traffic Indices are known or superseded by the City of Wildomar.

14.0 ADDITIONAL GEOTECHNICAL EVALUATION

Since geotechnical reports for the graded road fills along the eastern portion of the site, which may been constructed as part of the Elizabeth Lane improvements of the existing adjacent self storage facility were not available at the time of this investigation the existing fills should be further evaluated with additional subsurface exploration and laboratory testing prior to construction in order to provide appropriated earthwork recommendations, as necessary, relative to the proposed grading and development in the area, according the referenced 40-scale plot plan and preliminary grading plan.

15.0 PLAN REVIEWS AND CONSTRUCTION SERVICES

This report has been prepared for the exclusive use of The RANCON Group to assist the project engineer and architect in the design of the proposed development. It is recommended that LGC be engaged to review the rough grading plans, structural plans and the final design drawings and specifications prior to construction. This is to document that the recommendations contained in this report have been properly interpreted are incorporated into the project specifications. LGC's review of the rough grading plan may indicate that additional subsurface exploration, laboratory testing and analysis should be performed to address areas of concern. If LGC is not accorded the opportunity to review these documents, we can take no responsibility for misinterpretation of our recommendations.

We recommend that LGC be retained to provide geotechnical engineering services during both the rough grading and construction phases of the work. This is to document compliance with the design, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

If the project plans change significantly (e.g., building loads or type of structures), we should be retained to review our original design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appears to be different than those indicated in this report, this office should be notified immediately. Design and construction revisions may be required.

16.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable engineers and geologists practicing in this or similar localities. The professional opinions contained herein have been derived in accordance with current standards of practice. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report. The samples taken and submitted for laboratory testing, the observations made and the in-situ field testing performed are believed representative of the entire project; however, soil and geologic conditions can vary in characteristics between excavations, both laterally and vertically and may be different than our preliminary findings.

If this occurs, the changed conditions must be evaluated by the project geotechnical engineer and engineering geologist and design(s) adjusted as required or alternate design(s) recommended.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and/or project engineer and incorporated into the plans, and the necessary steps are taken to see that the contractor and/or subcontractor properly implements the recommendations in the field. The contractor and/or subcontractor should notify the owner if they consider any of the recommendations presented herein to be unsafe.

The conclusions and opinions contained in this report are based on the results of the described geotechnical evaluations and represent our professional judgment. The findings, conclusions and recommendations contained in this report are to be considered tentative only and subject to confirmation by the undersigned during the construction process. Without this confirmation, this report is to be considered incomplete and LGC or the undersigned professionals assume no responsibility for its use.

The conclusions and opinions contained in this report are valid up to a period of 2 years from the date of this report or a change within the California Building Code, which ever occurs first. Changes in the conditions of a property can and do occur with the passage of time, whether they be because of natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate codes or standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore if any of the above mentioned situations occur, an update of this report should be completed.

This report has not been prepared for use by parties or projects other than those named or designed above. It may not contain sufficient information for other parties or other purposes.

The opportunity to be of service is appreciated. Should you have any questions regarding the content of this report, or should you require additional information, please do not hesitate to contact this office at your earliest convenience.

APPENDIX A

REFERENCES AND AERIAL PHOTOGRAPHS

APPENDIX A

References Reviewed

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Aerial Photograph Interpretation Table

DATE	FLIGHT NUMBER	SCALE
9-11-97	C116-41-249-251	1" = 2,000'
1-29-95	15-14, 16-16, 16-17	1" = 1,600'
5-14-93	C91-6-7	1" = 2,000'
2-20-91	91033-12, 13	1" = 4,000'
1-13-89	890190-13, 14	1" = 4,000'
2-8-88	88045-10, 11, 15	1" = 4,000'
5-8-86	86119-7, 8	1" = 4,000'
7-15-67	AXM-3HH-76,77	1" = 1,667'
5-15-67	AXM-SHH-22, 23	1" = 1,667'
5-4-49	AXM-6F-10, 11	1" = 1,667'

APPENDIX B

BORING LOGS
(LGC INLAND, INC., 2005)

Geotechnical Boring Log B-3

Date: March 8, 2005	Project Name: R.S. Development - 45 Acres Wildomar — Page 1 of 1
Project Number: I04711-10	Logged By: AW
Drilling Company: 2R	Type of Rig: CME 55
Drive Weight (lbs): 140	Drop (in): 30 Hole Dia (in): 8
Top of Hole Elevation (ft): 1362±	Hole Location: See Geotechnical Map

Depth (ft)	Blow Count / 6"	Sample No.	Dry Density (pcf)	Moisture (%)	Geologic / USCS Symbol	DESCRIPTION	Type of Test
0					CL	***TOPSOIL:*** Sandy CLAY; dark brown, moist, medium stiff fine to medium sand	
	Bag 1 1-6'						
	50	R-1	116.6	12.2	Qps	***BEDROCK (PAUBA FORMATION):*** Sandy CLAYSTONE; red brown, moist, very hard, fine to coarse gravel	Atterberg
5	27 50 for 5½"	R-2				No Recovery	
	22 50 for 4"	R-3	126.5	10.4		Clayey SANDSTONE; light tan, slightly moist, very hard, fine to coarse sand	
10	50 for 4"	R-4	98.8	8.5		olive gray, moist	
15	27 35 43	S-1	–	6.4		yellowish brown, slightly moist	
20	55 for 4"	R-5	120.5	9.0	Kvt	***BEDROCK (VAL VERDE TONALITE):*** TONALITE; olive gray, slightly moist, very hard, extremely weathered, breaks down to SAND with clay	
25	17 33 47	S-2	–	11.5		Breakdown to clayey SAND	
						Total Depth: 26½ ***No Groundwater***	
30							

LGC INLAND

Geotechnical Boring Log B-4

Date: March 8, 2005	Project Name: R.S. Development - 45 Acres Wildomar — Page 1 of 1
Project Number: I04711-10	Logged By: AW
Drilling Company: 2R	Type of Rig: CME 55
Drive Weight (lbs): 140	Drop (in): 30 Hole Dia (in): 8
Top of Hole Elevation (ft): 1358±	Hole Location: See Geotechnical Map

Depth (ft)	Blow Count / 6"	Sample No.	Dry Density (pcf)	Moisture (%)	Geologic / USCS Symbol	DESCRIPTION	Type of Test
0					CL	***TOPSOIL:*** Sandy CLAY; yellowish brown, moist, medium stiff, fine to medium sand porosity	
	7 15 21	R-1	96.3	12.7	Qps	***BEDROCK (PAUBA FORMATION):*** Clayey SANDSTONE; reddish brown, moist, hard, fine to coarse sand	
5	10 16 21	R-2	123.9	12.2			
	12 21 24	R-3	118.0	9.1		slightly moist, fine to coarse sand, fine to coarse gravel	
10	21 27 30	R-4	122.4	10.7			
15	17 29 33	S-1	–	8.5		very hard	
						Total Depth: 16 ½ feet ***No Groundwater***	
20							
25							
30							

LGC INLAND

Geotechnical Boring Log B-5

Date: March 8, 2005	Project Name: R.S. Development - 45 Acres Wildomar — Page 1 of 2
Project Number: I04711-10	Logged By: AW
Drilling Company: 2R	Type of Rig: CME 55
Drive Weight (lbs): 140	Drop (in): 30 Hole Dia (in): 8
Top of Hole Elevation (ft): 1364±	Hole Location: See Geotechnical Map

Depth (ft)	Blow Count / 6"	Sample No.	Dry Density (pcf)	Moisture (%)	Geologic / USCS Symbol	DESCRIPTION	Type of Test
0					SC	***TOPSOIL:*** Clayey SAND; dark yellow brown, moist, firm, fine to coarse sand, porosity	
		Bag 1 1-6'					
	5 9 13	R-1	121.4	10.1	Qps	***BEDROCK (PAUBA FORMATION):*** Silty SANDSTONE; light tan, moist, moderately hard	Atterberg Max, EI
5	29 50 for 5"	R-2	106.9	21.3		Clayey SANDSTONE; light tan, moist, very hard, fine to coarse sand	R-value Sulfate
	19 33 50 for 3"	R-3	112.0	5.4		slightly moist	
10	25 50 for 4"	R-4	129.5	8.1			
						Sandy CLAYSTONE, tan, moist, medium hard, fine to coarse sand	
15	3 9 12	S-1	–	19.7			
20	15 32 50 for 4"	R-5	116.8	16.7		olive gray, moist, very hard, fine to medium sand	
25	17 32 41	S-2	–	11.4		fine to coarse sand	
30							

LGC INLAND

Geotechnical Boring Log B-5

Date: March 8, 2005	Project Name: R.S. Development - 45 Acres Wildomar Page 2 of 2
Project Number: I04711-10	Logged By: AW
Drilling Company: 2R	Type of Rig: CME 55
Drive Weight (lbs): 140	Drop (in): 30 Hole Dia (in): 8
Top of Hole Elevation (ft): 1364±	Hole Location: See Geotechnical Map

Depth (ft)	Blow Count / 6"	Sample No.	Dry Density (pcf)	Moisture (%)	Geologic / USCS Symbol	DESCRIPTION	Type of Test
30	20 50 for 5"	R-6	128.2	9.5		Clayey SANDSTONE; olive gray, slightly moist, very hard, fine to medium	
35	25 50 for 3"	S-3	-	13.4		Sandy CLAYSTONE; olive gray, moist, very hard, fine to medium sand	
						Clayey SANDSTONE; olive gray, moist, very hard, fine to medium sand	
40	36 50 for 5"	R-7	122.0	14.0			
						Total Depth: 41 feet	
						No Groundwater	
45							
50							
55							
60							

LGC INLAND

Geotechnical Boring Log B-6

Date: March 8, 2005	Project Name: R.S. Development - 45 Acres Wildomar — Page 1 of 1
Project Number: I04711-10	Logged By: AW
Drilling Company: 2R	Type of Rig: CME 55
Drive Weight (lbs): 140	Drop (in): 30 — Hole Dia (in): 8
Top of Hole Elevation (ft): 1357±	Hole Location: See Geotechnical Map

Depth (ft)	Blow Count / 6"	Sample No.	Dry Density (pcf)	Moisture (%)	Geologic / USCS Symbol	DESCRIPTION	Type of Test
0					CL	***TOPSOIL:*** Sandy CLAY; dark yellow brown, moist, firm, rootlets	
					Qps	***BEDROCK (PAUBA FORMATION):***	
	17 23 50 for 5½"	R-1	115.2	15.3		Sandy CLAYSTONE; medium brown, moist, very hard, fine to coarse sand	
5	29 50	R-2	127.9	10.9		fine to medium sand	
10	18 21 23	R-3	123.6	9.7		Clayey SANDSTONE; red yellow, slightly moist, hard, fine to coarse sand	
15	10 17 18	R-4	110.9	13.2		CLAYSTONE; light brown, moist, moderately hard	
20						***Total Depth: 16½ feet*** ***No Groundwater***	
25							
30							

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Geotechnical Boring Log B-7

Date: March 8, 2005	Project Name: R.S. Development - 45 Acres Wildomar — Page 1 of 1
Project Number: I04711-10	Logged By: AW
Drilling Company: 2R	Type of Rig: CME 55
Drive Weight (lbs): 140	Drop (in): 30 — Hole Dia (in): 8
Top of Hole Elevation (ft): 1373±	Hole Location: See Geotechnical Map

Depth (ft)	Blow Count / 6"	Sample No.	Dry Density (pcf)	Moisture (%)	Geologic / USCS Symbol	DESCRIPTION	Type of Test
0					CL	***TOPSOIL:*** Sandy CLAY; dark brown, moist, firm	
		Bag 1 2-7'			Qps	***BEDROCK (PAUBA FORMATION):***	
	16 21 23	R-1	126.0	12.3		Clayey SANDSTONE; medium brown, moist, hard, fine to coarse sand	
5	12 21 46	R-2	111.9	18.5		Sandy CLAYSTONE; medium brown, moist, very hard, fine to medium sand	
	12 14 19	R-3	122.7	10.9		Clayey SANDSTONE; olive gray, moist, medium hard, fine to coarse sand	
10	12 21 31	R-4	111.6	6.1		slightly moist	
						Total Depth:11½ feet	
						No Groundwater	
15							
20							
25							
30							

LGC INLAND

APPENDIX C

LABORATORY TESTING PROCEDURES AND TEST RESULTS (LGC INLAND, INC., 2005)

. APPENDIX C

Laboratory Testing Procedures and Test Results

The laboratory testing program was directed towards providing quantitative data relating to the relevant engineering properties of the soils. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and a table summarizing the test results.

Soil Classification: Soils were classified in general accordance with ASTM Test Methods D2487 and D2488. This system utilizes the Atterberg limits and grain size distribution of a soil. The soil classifications (or group symbol) are shown on the laboratory test data, boring logs and trench logs.

Moisture and Density Determination Tests: Moisture content (ASTM D2216) and dry density determinations (ASTM D2937) were performed on relatively undisturbed samples obtained from the test borings and/or trenches. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

Maximum Density Tests: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM D1557. The results of these tests are presented in the table below:

SAMPLE LOCATION	*SAMPLE DESCRIPTION*	*MAXIMUM DRY DENSITY (pcf)*	*OPTIMUM MOISTURE CONTENT (%)*
B-3 @ 1-6 feet	Clayey SAND/Sandy CLAY	126.0	9.5

Expansion Index: The expansion potential of selected samples was evaluated by the Expansion Index Test ASTM D4829. Specimens are molded under a given compactive energy to approximately the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch thick by 4-inch diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below:

SAMPLE LOCATION	*SAMPLE DESCRIPTION*	*COMPACTED DRY DENSITY (pcf)*	*EXPANSION INDEX*	*EXPANSION POTENTIAL**
B-3 @ 1-6 feet	Clayey SAND/Sandy CLAY	104.2	18	VERY LOW

* Per ASTM D4829

Atterberg Limits: The liquid and plastic limits ("Atterberg Limits") were determined in accordance with ASTM Test Method D4318 for engineering classification of fine material and presented in the table below:

SAMPLE LOCATION	*LIQUID LIMIT*	*PLASTIC LIMIT*	*PLASTICITY INDEX*	*USCS SOIL SYMBOL*
B-3 @ 1-6 feet	32	15	17	SC/CL

Soluble Sulfates: The soluble sulfate contents of selected sample(s) were determined by standard geochemical methods (CTM 417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the table below:

SAMPLE LOCATION	*SAMPLE DESCRIPTION*	*SULFATE CONTENT (% by weight)**	*SULFATE EXPOSURE**
B-3 @ 1-6 feet	Clayey SAND/Sandy CLAY	0.002	Negligible

* Per ACI 318R-05 Table 4.3.1

R-Value: The resistance R-value was determined by the ASTM D2844 soils. The sample(s) were prepared and exudation pressure and R-value were determined. These/This result(s) were used for asphaltic concrete pavement design purposes.

SAMPLE LOCATION	*SAMPLE DESCRIPTION*	*R-VALUE*
B-3 @ 1-6 feet	Clayey SAND/Sandy CLAY	13

APPENDIX D

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

APPENDIX D

LGC Geo-Environmental, Inc.

General Earthwork and Grading Specifications for Rough Grading

1.0 General

1.1 ***Intent:*** These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 ***The Geotechnical Consultant of Record:*** Prior to commencement of work, the owner shall employ a qualified Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to confirm that the attained level of compaction is being accomplished as specified. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 ***The Earthwork Contractor:*** The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and

processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the project plans and specifications. The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "equipment" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading.
The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate personnel will be available for observation and testing. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified. It is the contractor's sole responsibility to provide proper fill compaction.

2.0 *Preparation of Areas to be Filled*

2.1 ***Clearing and Grubbing:*** Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 10 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed. The contractor is responsible for all hazardous waste relating to his work. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Client should acquire the services of a qualified environmental assessor.

2.2 ***Processing:*** Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 ***Overexcavation:*** In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 ***Benching:*** Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 ***Evaluation/Acceptance of Fill Areas:*** All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 *Fill Material*

3.1 ***General:*** Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 ***Oversize:*** Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical

Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 ***Import:*** If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 *Fill Placement and Compaction*

4.1 ***Fill Layers:*** Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 ***Fill Moisture Conditioning:*** Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

4.3 ***Compaction of Fill:*** After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 ***Compaction of Fill Slopes:*** In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 ***Compaction Testing:*** Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

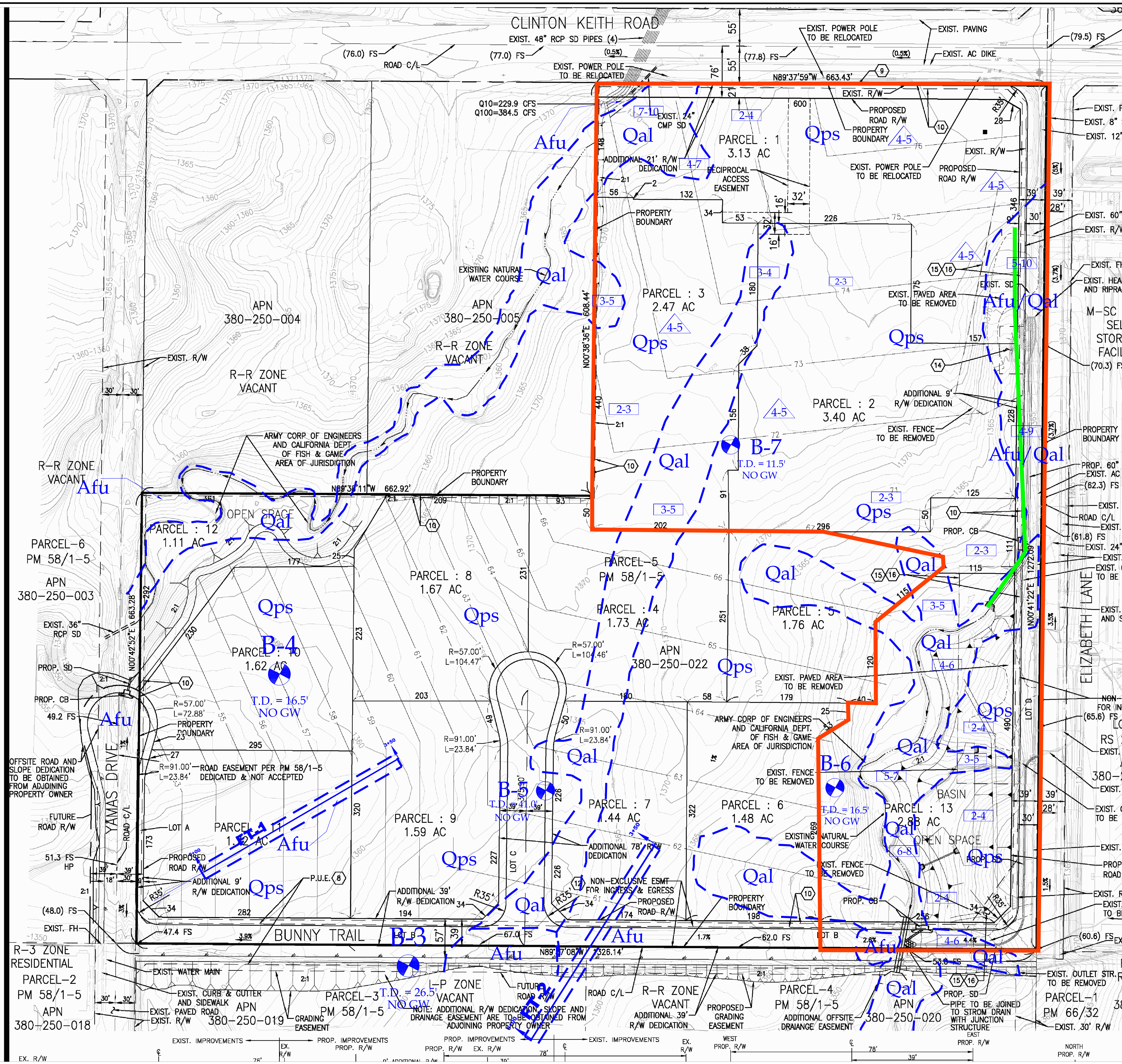
4.6 ***Frequency of Compaction Testing:*** Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one (1) test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 ***Compaction Test Locations:***

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two (2) grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 ***Subdrain Installation***

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrain and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.



LEGEND

(Locations are Approximate)

Earth Units

- Afu - Artificial Fill, Undocumented
- Qal - Quaternary Alluvium
- Qps - Quaternary Pauba Formation

Symbols

- Limits of Report Pertaining to 40-scale Plot Plan and Preliminary Grading Plan for Medical and Education Center
- Geologic Contact
- B-7 — Boring Location (LGC, 2005)
- FT-1 & FT-2 — Fault Trench Location (LGC, 2005)
- 7-10 — Estimated Overexcavation Depth of Unsuitable Materials Below Existing Grade (ft)
- 4-5 — Estimated Overexcavation Depth Below Proposed Grade In Cut Areas (ft)
- Tentative Subdrain Location

LGC

LGC
GEO-ENVIRONMENTAL, INC.
31915 RANCHO CALIFORNIA ROAD
TEMECULA, CALIFORNIA 92591
Office: (951) 297-2450
Fax: (951) 344-8214

Robert Gregorek II — Engineering Geologist
Larry D. Cooley — Project Engineer

GEOTECHNICAL MAP

TENTATIVE PARCEL MAP NO. 36492
CITY OF WILDOMAR, COUNTY OF RIVERSIDE, CALIFORNIA

Name:	RANCON MEDICAL CENTER
Project No.	G04-711-10
Client:	THE RANCON GROUP
Scale:	1"=50'
Date:	AUGUST 2012
Reference:	80-SCALE TENTATIVE PARCEL MAP NO. 36492
Plate No.	1

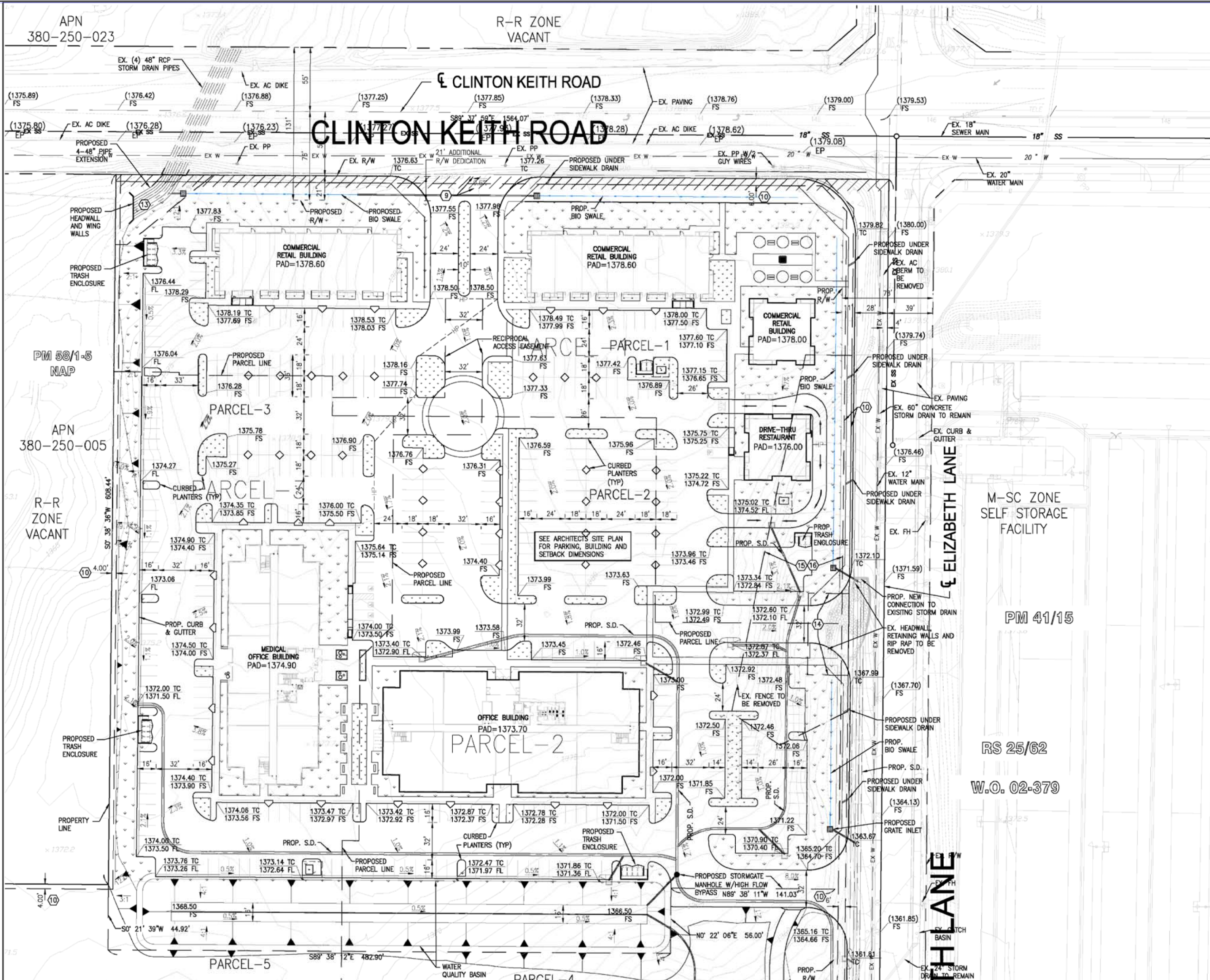


PLATE 2

SCALE:	1"=40'
DATE:	07/16/12
DESIGNED:	MB
CHECKED:	
PLN CK REF:	
F.B.	

ALBERT A. WEBB ASSOCIATES

ENGINEERING CONSULTANTS
3788 McCRAY STREET
RIVERSIDE, CA. 92506
PH. (951) 686-1070
FAX (951) 788-1256

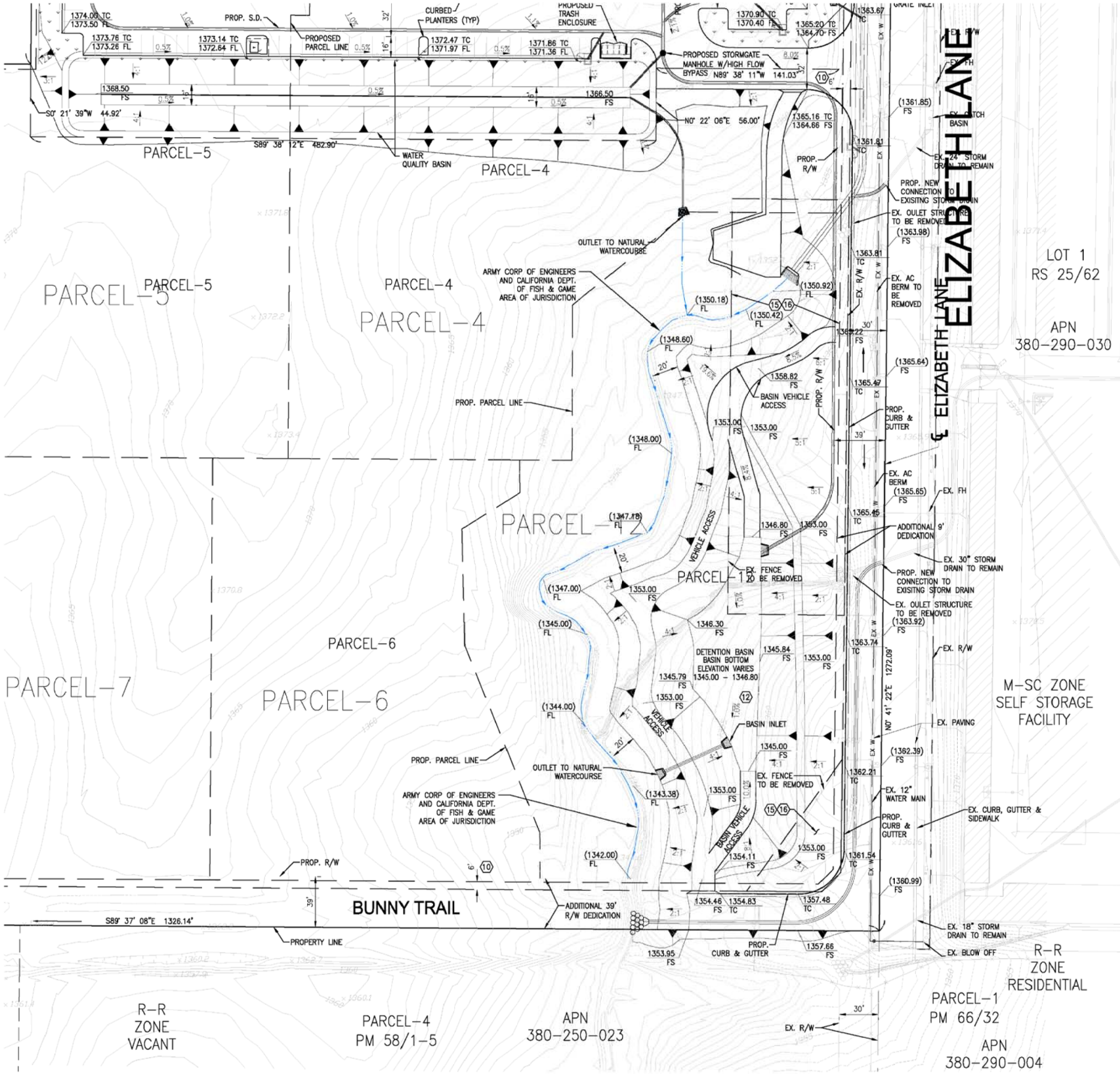
PLOT DATE: 30-Jul-12

PLOT PLAN
CITY OF WILDOMAR
PRELIMINARY GRADING PLAN
TENTATIVE PARCEL MAP NO. 36492

RANCON MEDICAL & EDUCATIONAL CENTER

W.O. 12-0031
SHEET 2 OF 3 SHEETS
DWG. NO.

REVISIONS | DATE | BY



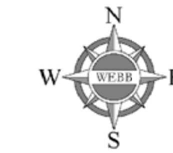


PLATE 3

		SCALE: 1"=40'	ALBERT A. WEBB ASSOCIATES	ENGINEERING CONSULTANTS 3788 McCRAY STREET RIVERSIDE, CA. 92506 PH. (951) 686-1070 FAX (951) 788-1256	PLOT PLAN EDUCATIONAL CENTER PRELIMINARY GRADING PLAN TENTATIVE PARCEL MAP NO. 36492	W.O. 12-0031
		DATE: 07/16/12				SHEET 3
		DESIGNED: MB				OF 3 SHEETS
		CHECKED: WILDOMAR				
		PLN CK REF:		PLOT DATE: 30-Jul-12	RANCON MEDICAL & EDUCATIONAL CENTER	DWG. NO.
REVISIONS	DATE	BY	F.B.			