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GEOTECHNICAL INVESTIGATION

OTAY RANCH RESORT VILLAGE
OTAY LAKES ROAD
WIDENING AND REALIGNMENT
SAN DIEGO COUNTY, CALIFORNIA

PREPARED FOR

BALDWIN & SONS, LLC
SAN DIEGO, CALIFORNIA

OTAY RANCH, LLC, & JPB DEVELOPMENT, LLC
CHULA VISTA, CALIFORNIA

NOVEMBER 11, 2014
PROJECT NO. G1012-52-01A

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Project No. G1012-52-01A
November 11, 2014

Baldwin & Sons, LLC
610 West Ash Street, Suite 1500
San Diego, California 92101

Attention: Mr. Scott Molloy

Otay Ranch, LLC, & JPB Development, LLC
1392 East Palomar Street, Suite 202
Chula Vista, California 91913

Attention: Mr. Sean Kilkenny

Subject: GEOTECHNICAL INVESTIGATION
OTAY RANCH RESORT VILLAGE
OTAY LAKES ROAD WIDENING AND REALIGNMENT
SAN DIEGO COUNTY, CALIFORNIA

Dear Mr. Molloy and Mr. Kilkenny:

In accordance with your request, we are submitting the results of our geotechnical investigation for the Otay Lakes Road widening and realignment project associated with Otay Ranch Resort Village. The accompanying report presents the findings, conclusions, and recommendations from our study. Based on the results of this study, it is our opinion that the proposed roadway improvements can be constructed as planned provided the recommendations of this report are followed.

If you have any questions regarding this investigation, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

We prepared this geotechnical investigation for the proposed improvements along a portion of Otay Lakes Road. The improvements start on the west from Lake Crest Drive and extend eastward adjacent to a private airport facility in San Diego County, California (see Vicinity Map, Figure 1). The length of the planned improvements is approximately 4 miles. The western approximately 1,200 feet of the roadway improvement is within the City of Chula Vista and the eastern project limits is about 3 miles west of the City of Chula Vista. We understand the roadway improvements will be performed in conjunction with the development of the adjacent Otay Ranch Resort Village project located along the north side of the roadway. The improvements include approximately 22,000 lineal feet of road widening/realignment and the construction of four retaining walls and two animal under-crossings. The adjacent residential development proposes to construct seven storm-water-quality basins on the north side of the roadway with related storm-drain pipes and structures.

The purpose of this investigation is to evaluate the alignment's surface and subsurface soil conditions, general site geology and geotechnical constraints, and to provide recommendations pertinent to the geotechnical engineering aspects of improving the roadway as proposed including providing design recommendations for the proposed animal under crossings and retaining walls.

The scope of our investigation included a site reconnaissance, a field investigation, laboratory testing, engineering analyses, slope stability analyses, and preparation of this report. We performed a field exploration that consisted of drilling 10 small diameter borings within the area of the existing roadway with a truck-mounted drill rig. In addition, we performed four large diameter borings, nine excavator trenches, one air-track boring, and 22 backhoe trenches in the area of the roadway realignment within adjacent slopes and canyon drainages. Three backhoe trenches performed by Neblett and Associates (2004) are also included. We performed the field investigation to examine and sample the soil and geologic units within areas of the proposed improvement including the animal under crossings, retaining walls and storm drain pipelines. Logs of the exploratory borings, trenches, and other details of the field investigation are presented in Appendix A. We performed our investigation for the roadway concurrently with the investigation for the Otay Ranch Resort Village; therefore, the exploration numbering sequence is not in numerical order.

We performed laboratory tests on selected soil samples obtained from the borings and trenches to evaluate their pertinent physical and chemical properties for engineering analyses. A discussion of the laboratory testing and results is presented in Appendix B and on the boring and trench logs in Appendix A. The conclusions and recommendations presented herein are based on an analysis of the data obtained from the exploratory borings, trenches, laboratory tests, and our experience with similar soil and geologic conditions.

2. SITE AND PROJECT DESCRIPTION

The project extends from Lake Crest Drive approximately 22,000 feet to the east. Approximately 600 feet of Wueste Road will be improved at the intersection of Otay Lakes Road. The portion of Otay Lakes Road proposed for widening and realignment traverses along the north side of Lower Otay Lake. Upper Otay Lake is located to the north of the roadway and the western portion of the project. To accommodate the proposed widening and realignment of the roadway, additional grading consisting of constructing cut and fill slopes with a maximum height of approximately 70 and 40 feet high, respectively, at a maximum inclination of 2:1 (horizontal to vertical) is proposed. In addition, four retaining walls ranging in height from approximately 15 to 25 feet are proposed at two locations along both sides of Otay Lakes Road. The purpose of the walls is to provide lateral support to the roadway in conjunction with the installation of two pre-cast concrete animal under crossings proposed within canyon drainages. The walls may be constructed as mechanically stabilized earth (MSE) segmental retaining walls or CMU retaining walls.

Otay Lakes Road is currently paved with three inches of asphalt concrete with no underlying aggregate base material based on our observations. Surface elevations of the existing roadway range from a high of approximately 589 feet above Mean Sea Level (MSL) at the western edge of the improvement to approximately 516 feet MSL at the eastern edge with a low of approximately 504 feet MSL at several locations along the alignment.

3. SOIL AND GEOLOGIC CONDITIONS

During our field investigation, we encountered six surficial deposits, consisting of undocumented fill, topsoil, lacustrine deposits, alluvium, colluvium and older alluvium, and three geologic formations, consisting of sedimentary Tertiary-age Fanglomerate Deposits and Tertiary-age Otay Formation, and Jurassic to Cretaceous-age Metavolcanic Rock. The materials encountered in each excavation are shown on the Geologic Map, Figures 2 through 8 (map pocket). Geologic Cross Sections prepared as part of the adjacent Otay Ranch Resort Village investigation that extended into the roadway are presented on Figures 9 through 11 (map pocket). The descriptions of the soil and geologic conditions are shown on the boring and trench logs located in Appendix A and are described herein in order of increasing age.

3.1 Undocumented Fill (Qudf)

We encountered undocumented fill in 9 of the 10 exploratory borings. The undocumented fill is generally located in areas where the roadway crosses canyon drainages at 14 locations. The fill is characterized as medium dense, silty to clayey sand and firm to stiff, sandy clay with varying amounts of gravel and cobble. The fill soil, as encountered in our exploratory excavations, ranged in thickness from approximately 5 to 25 feet. The undocumented fill in our opinion is not prone to significant settlement due to the lack of additional fill loading and based on our review of the laboratory consolidation test results. Remedial grading will not be required within the existing undocumented fill with the exception of the upper subgrade surface of the pavement section that will be prepared during paving operations. However, benching of the existing fill slopes will be required in areas of road widening.

3.2 Topsoil (unmapped)

Holocene-age topsoil is present as a thin veneer locally overlying formational materials across the site. The topsoil has an average thickness of approximately 3 feet and can be characterized as soft to stiff and loose to medium dense, dry to damp, dark brown, sandy clay to clayey sand with gravel and cobble. The topsoil is typically expansive and compressible. Removal of the topsoil will be necessary in areas to support proposed fill or structures. Due to the relatively thin thickness and discontinuity of these deposits, topsoil is not shown on the Geologic Map.

3.3 Lacustrine Deposits (Ql)

The areas of the Upper and Lower Otay Lakes and the canyon drainage between the two lakes are underlain by Lacustrine Deposits. Lacustrine Deposits are sediments derived from the surrounding landforms deposited at the bottom or adjacent to the existing lakes. The Lacustrine Deposits will be encountered within the canyon drainage between the two lakes on the western portion of the roadway and will require remedial grading in areas to support proposed fill or structures. This material will be saturated and difficult to excavate and reuse as fill soil.

3.4 Alluvium (Qal)

Holocene-age alluvium is sheet-flow or stream deposited material found within the canyon drainages and generally vary in thickness dependent upon the size of the canyon and extent of the drainage area. The alluvium beneath the undocumented fill consists of firm to very stiff, light brown to reddish brown, sandy clay and medium dense to very dense, brown to yellowish brown, clayey sand with gravel and cobble. The alluvium within the canyon drainages not overlain by undocumented fill is loose to medium dense and can become saturated and difficult to excavate during the rainy season. The thickness of the alluvium encountered in our exploratory excavations and in previous trenches excavated by Neblitt & Associates (2004) ranged up to approximately 10 feet and may be thicker in the larger canyon drainages. Due to the relatively unconsolidated nature of these deposits, remedial grading will be necessary in areas to receive proposed fill or structures. The alluvium located beneath the undocumented fill will not require remedial grading.

3.5 Colluvium (unmapped)

Holocene-age colluvium, derived from weathering of the underlying bedrock materials at higher elevations and deposited by gravity and sheet-flow, is locally present on the side slopes of canyon drainages. The colluvium can be characterized as sandy clay and clayey sand with varying amounts of gravel and cobble. The thickness of colluvium generally ranges from approximately 2 to 5 feet, but can be thicker along the lower portions of canyons and toes of natural slopes. Due to the relatively thin thickness and discontinuity of these deposits, the colluvium is not shown on the geologic map. Remedial grading of the colluvium will be required in areas that will support proposed fill or structures.

3.6 Older Alluvium (Qoal)

Pleistocene-age older alluvium is located south of the roadway along a portion of the eastern limits of the project. This unit typically consists of dark brown, dense, slightly cemented, clayey sand and sandy clay with some gravel and cobble. This unit will not be encountered during site improvements with the possible exception of the eastern most canyon drainage. We do not expect remedial grading of the unit would be required.

3.7 Fanglomerate Deposits (Tof)

Tertiary-age Fanglomerate Deposits are located along several portions of the roadway. This unit typically consists of dense to very dense, moderately cemented, clayey and silty sandstone and occasional sandy claystone with subrounded to subangular cobbles and boulders up to 40 percent generally up to 2 feet in diameter. Excavations within this unit will likely be very difficult and require specialized heavy-duty equipment. Oversized material will likely be generated during grading operations within the cemented portions. The Fanglomerate Deposits are generally suitable for the support of compacted fill and structural loads and are generally stable when excavated to construct cut slopes.

3.8 Otay Formation (To)

Tertiary-age Otay Formation is located along most of the roadway alignment. This unit consists of dense to very dense and hard, slightly and moderately cemented, clayey sandstone and sandy claystone with interbeds of gravel and cobble generally with a maximum dimension of 1 foot. In addition, localized and discontinuous areas of sheared claystone bedding can occur within this unit that can create slope instability. Excavations within the unit will generally be possible with heavy-duty equipment; however, moderately cemented zones will create very difficult ripping and generate oversize material. The Otay Formation is suitable for the support of proposed fill and structural loads. In addition, the sandstone portions of this unit are generally stable when excavated to construct cut slopes. The sheared claystone layers may require slope stabilization if encountered in cut slopes.

3.9 Metavolcanic Rock (KJmv)

Metavolcanic Rock mapped by the California Geologic Survey (2002) as Cretaceous to Jurassic in age (formerly known as the Santiago Peak Volcanics) is present on the western and eastern portions of the roadway. The metavolcanic rock is generally moderately strong to strong, highly to slightly weathered, and jointed. Highly weathered portions of the Metavolcanic Rock are generally rippable with heavy-duty grading equipment and will generate clayey and sandy soil with cobble and boulder-size rock. Moderately to slightly weathered Metavolcanic Rock will generate oversize rock material and will likely be very difficult to excavate or unrippable and, if encountered within cut areas, will likely require blasting or rock breaking to excavate. The Metavolcanic Rock is considered stable for construction of the proposed cut slopes if free of loose rock after blasting and excavation. Areas of rock fall mitigation of adjacent open space areas will be required as discussed in the *Rock Fall Hazard* section of the report.

4. GROUNDWATER

We did not encounter groundwater during our field investigation within the small-diameter borings within the roadway to the maximum depth explored of 37½ feet. However, seepage was encountered perched on the alluvium or formational material in Borings SB-7 and SB-9 and in several trenches within the canyon drainages. Groundwater is expected to affect construction and remedial grading within the Lacustrine Deposits in the canyon area between the two lakes. We also expect perched groundwater or seepage to have some impact in areas that will require alluvium removal within the other canyon drainages especially during the rainy season. It is not uncommon for groundwater or seepage conditions to develop where none previously existed. Groundwater elevations are dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary as a result. Proper surface drainage will be important to future performance of the project.

5. GEOLOGIC HAZARDS

5.1 Landslides

Landslides are not known to exist or mapped in the area of improvement and we did not encounter landslides along the roadway alignment during our field investigation. We do not consider the potential for landsliding to be a hazard to the project.

5.2 Rock Fall Hazard

Due to the steep terrain along with localized areas of large boulder outcrops above the eastern portion of the roadway, the potential hazard for future rock fall is a consideration for development. We evaluated the natural slopes for their potential rock fall impact to the proposed roadway by performing field mapping of the open space natural rock slopes. Based on our experience with rock fall potential, mitigation measures will be required along the eastern portion of the roadway due to the steepness of the natural slopes and boulder outcrops above the proposed cut slope. The area of proposed rock fall mitigation is shown on the Geologic Map, Figure 8. The mitigation may consist of the construction of a rock fall debris fence or other acceptable catchment devise at the toe of the proposed cut slope. The hard rock slopes should be evaluated by an engineering geologist during site development and final locations of the debris fence or alternative method can be provided at that time.

5.3 Faulting and Seismicity

A review of the referenced geologic materials and our knowledge of the general area indicate that the site is not underlain by active, potentially active, or inactive faults. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,000 years. The site is not located within a State of California Earthquake Fault Zone.

According to the computer program *EZ-FRISK* (Version 7.62), 7 known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database that provides several models and combinations of fault data to evaluate the fault information. Based on this database, the nearest known active fault is the Newport-Inglewood and Rose Canyon Fault system, located approximately 14 miles west of the site, and is the dominant source of potential ground motion. Earthquakes that might occur on the Newport-Inglewood and Rose Canyon Faults or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood Fault are 7.5 and 0.20g, respectively. Table 5.3.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2007) NGA USGS2008 acceleration-attenuation relationships.

TABLE 5.3.1
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2007 (g)
Newport - Inglewood	14	7.5	0.20	0.16	0.20
Rose Canyon	14	6.9	0.16	0.14	0.14
Coronado Bank	22	7.4	0.14	0.11	0.12
Palos Verdes Connected	22	7.7	0.16	0.12	0.15
Elsinore	37	7.9	0.12	0.08	0.10
Earthquake Valley	41	6.8	0.06	0.05	0.04
San Jacinto	49	7.9	0.09	0.07	0.08

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2007) NGA USGS2008 in the analysis. Table 5.3.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 5.3.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2007 (g)
2% in a 50 Year Period	0.32	0.32	0.35
5% in a 50 Year Period	0.23	0.23	0.24
10% in a 50 Year Period	0.17	0.17	0.17

The California Geologic Survey (CGS) has a program that calculates the ground motion for a 10 percent of probability of exceedence in 50 years based on an average of several attenuation relationships. Table 5.3.3 presents the calculated results from the Probabilistic Seismic Hazards Mapping Ground Motion Page from the CGS website.

TABLE 5.3.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Soft Rock	Calculated Acceleration (g) Alluvium
0.21	0.23	0.27

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structure should be evaluated in accordance with the California Building Code (CBC) guidelines currently adopted by the City of Chula Vista and County of San Diego.

5.4 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless, groundwater is encountered within 50 feet of the surface, and soil relative densities are less than about 70 percent. If the four previous criteria are met, a seismic event could result in a rapid pore-water pressure increase from the earthquake-generated ground accelerations. Seismically induced settlement may occur whether the potential for liquefaction exists or not. The potential for liquefaction and seismically induced settlement occurring within the site soil is considered to be very low due to the medium dense nature of the existing undocumented fill and underlying alluvium and the very dense nature of the formational materials.

5.5 Hydroconsolidation

Hydroconsolidation is the tendency of unsaturated soil structure to collapse upon saturation resulting in the overall settlement of the affected soil and any overlying foundations or improvements supported thereon. Compressible undocumented fill soil left in-place within the existing roadway possesses a potential for settlement due to hydroconsolidation. Based on the laboratory test results, the potential for hydroconsolidation of the undocumented fill will range from 0 to about 6.25 percent. We expect several inches of settlement could occur due to hydroconsolidation if water infiltrates the undocumented fill.

5.6 Tsunamis and Seiches

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water after a significant ocean or near shore earthquake. The site is approximately 11 miles from the Pacific Coast and ranges between approximately 500 feet and 590 feet above MSL. Because the site is not located near the ocean, the potential impact to the site by tsunamis is considered negligible.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement or lateral ground movement during significant seismic ground accelerations from an earthquake. The roadway is located along the north side of Lower Otay Lake and is roughly 30 to 120 feet above the current water level. The potential for a seiche to occur is considered moderate due to the potential for lateral ground motion to create a surface wave during a seismic event. However, it is our opinion that the impact to the roadway will be minimal due to the elevation difference of the roadway. Some minor erosion of the south facing slopes of the roadway may occur if water impacts the face of the slopes; however, it is our opinion that it will not impact the roadway pavement. The potential for a seiche to occur on the Upper Otay Lake is also moderate with the potential impact to the roadway consisting of water overtopping the concrete dam and flowing down the canyon drainage roughly 600 feet and impacting the north facing fill slope. The proposed slope has a height of roughly 30 feet, which creates a basin roughly 600 feet by 150 feet. It is our opinion that the basin area is of sufficient size to accommodate water that is generated from overtopping the dam during a seiche event. Therefore, the impact to the roadway is considered low.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No soil or geologic conditions were encountered at the site that, in the opinion of Geocon Incorporated, would preclude construction of the roadway improvements as planned provided the recommendations of this report are followed.

6.1.2 With the exception of possible seismic shaking and rock fall, we did not observe significant geologic hazards and do not know of any geologic hazards to exist on the site that would adversely affect the proposed roadway improvement. Special seismic design considerations other than those recommended herein are not required.

6.1.3 The area of the proposed roadway improvement is underlain by localized areas of surficial soil consisting of up to approximately 35 feet of undocumented fill and alluvium overlying the Fanglomerate Deposits, Otay Formation and/or Metavolcanic Rock. The undocumented fill and underlying alluvium is considered suitable for support of the roadway improvements. The surficial soil within canyon drainages consisting of alluvium, topsoil, lacustrine deposits, and colluvium that will be encountered during roadway widening and realignment is considered unsuitable for support of proposed fill soil or structural improvements and remedial grading will be necessary.

6.1.4 We did not encounter groundwater conditions during our subsurface exploration. However, we did encounter seepage conditions that were perched on the alluvium or formational materials within two of the exploratory borings within the roadway and within the canyon drainages within the alluvium. We expect groundwater to impact the grading within the canyon area between the two lakes where the roadway will be widened and fill slopes will extend into Lacustrine Deposits. The groundwater impact will be limited to remedial grading operations, however, groundwater will not impact the performance of the roadway once constructed.

6.1.5 We understand pre-cast reinforced concrete animal under crossings and mechanically stabilized earth (MSE) and/or CMU retaining walls may be used for this project. Geotechnical engineering recommendations for the retaining walls and concrete structures are presented herein. The walls should be founded on medium dense, undocumented fill and/or the underlying alluvium, dense formational materials, or proposed compacted fill.

6.1.6 In general, the Metavolcanic Rock is highly weathered near the surface to depths of 3 to 9 feet based on our exploratory excavations. However, the degree and depth of weathering can be highly variable, and very difficult excavation or refusal should be expected within moderately to slightly weathered rock. The contractor should expect some heavy ripping and the use of hydraulic rock breakers or blasting if resistant rock is encountered. The geologic map, Figures 2 through 8, presents the areas we expect rock will be encountered during grading.

6.2 Soil and Excavation Characteristics

6.2.1 The on-site surficial soil includes undocumented fill and underlying alluvium within the roadway prism, and alluvium, colluvium, lacustrine deposits and topsoil within the canyon drainages proposed for roadway widening and realignment. The surficial soil generally consists of clayey sand and sandy clay with varying amounts of gravel and cobble. The formational units that will be encountered in cut areas consist of slightly and moderately cemented sedimentary Fanglomerate Deposits and Otay Formation, and highly to slightly weathered Metavolcanic Rock. The sedimentary formational units generally consist of clayey sandstone and siltstone with gravel, cobble and boulders varying in size from 3 inches to 2 feet. The rock materials will excavate as clayey and sandy soil with rock fragments within the highly weathered areas to fractured rock material within the less weathered portions.

6.2.2 The surficial soil can be excavated with light to moderate effort using conventional heavy duty grading equipment. Wet conditions should be expected within the canyon drainages especially during the rainy season, which will create difficult excavation procedures. The greatest difficulty will likely be in the canyon area between the two lakes where top loading of wet soil may be necessary and extensive drying and mixing required. The sedimentary formational units can likely be excavated with heavy effort and will likely generate some oversize cemented chunks that will require special handling. The highly weathered portions of the rock can be excavated using heavy grading equipment with heavy effort. Very difficult excavation will be encountered in less weathered rock. Rock breaking or blasting may be necessary in hard rock areas if encountered during grading or utility excavations. The Geologic Map, Figures 2 through 8, presents the areas we expect rock may be encountered during grading and excavating operations.

6.2.3 The soil encountered in the field investigation is considered to be “expansive” (expansion index [EI] greater than 20) as defined by 2013 California Building Code (CBC) Section 1803.5.3. Table 6.2.1 presents soil classifications based on the expansion index. Based on laboratory test results of representative samples of the materials expected to be encountered and our experience with similar soil condition on the adjacent project to the north, the majority of the on-site material is expected to possess a “very low” to “high” expansion potential (EI of 130 or less). The surficial soil and the siltstone and claystone layers within the formational materials will likely have a “medium to “high” expansion potential (EI of 51 to 130). The sandstone layers within the formational materials will likely have a “very low” to “low” expansion potential (EI of 50 or less). Soil with a “medium” to “high” expansion potential (expansion index of greater than 50) should not be used as backfill material for the proposed retaining walls or placed within 3 feet of pavement grade.

TABLE 6.2.1
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2013 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

6.2.4 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B and indicate that the on-site materials at the locations tested possess “Not Applicable” (S0) to “Moderate” (S1) sulfate exposure, respectively, to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3. Table 6.2.2 presents a summary of concrete requirements set forth by 2013 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 6.2.2
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Severity	Exposure Class	Water-Soluble Sulfate (SO_4) Percent by Weight	Cement Type (ASTM C 150)	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Not Applicable	S0	$SO_4<0.10$	--	--	2,500
Moderate	S1	$0.10\leq SO_4<0.20$	II	0.50	4,000
Severe	S2	$0.20\leq SO_4\leq 2.00$	V	0.45	4,500
Very Severe	S3	$SO_4>2.00$	V+Pozzolan or Slag	0.45	4,500

6.2.5 We performed Potential of Hydrogen (pH) and resistivity, and water-soluble chloride ion content testing on selected samples to evaluate the corrosion potential to subsurface metal structures. We performed the tests in accordance with California Test Method 643 and AASHTO Test No. T291, respectively. The results are presented in Appendix B. These test results should be reviewed and incorporated into the design of buried metal structures.

6.2.6 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, if improvements that could be susceptible to corrosion are planned, further evaluation by a corrosion engineer should be performed.

6.3 Seismic Design Criteria

6.3.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 6.3.1 summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The improvements should be designed using a Site Class C. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2013 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 6.3.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 6.3.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	C	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.807g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.313g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.077	Table 1613.3.3(1)
Site Coefficient, F_V	1.487	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.869g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.465g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.579g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.310g	Section 1613.3.4 (Eqn 16-40)

6.3.2 Table 6.3.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 6.3.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.307g	Figure 22-7
Site Coefficient, F_{PGA}	1.093	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.336g	Section 11.8.3 (Eqn 11.8-1)

6.3.3 Conformance to the criteria in Tables 6.3.1 and 6.3.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.4 Temporary Slopes Recommendations

6.4.1 The recommendations presented herein are provided for the contractor to create stable excavations. It is the responsibility of the contractor and their “competent person” to provide a safe excavation during the construction of the proposed project.

6.4.2 Temporary slopes should be made in conformance with OSHA requirements. Surficial materials consisting of lacustrine deposits, undocumented fill, alluvium, colluvium, topsoil, older alluvium and proposed fill should be considered a Type B soil (Type C soil if seepage or groundwater is encountered) in accordance with OSHA requirements. Formational materials consisting of Fanglomerate Deposits, Otay Formation and Metavolcanic Rock can be considered a Type A soil (Type B soil if seepage or groundwater is encountered). In general, no special shoring requirements will be necessary if temporary excavations will be less than 4 feet high. Temporary excavation depths greater than 4 feet, however, should be laid back at an appropriate inclination. These excavations should not be allowed to become saturated or dry out. Surcharge loads should not be permitted within a distance equal to the depth of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended by OSHA or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

6.5 Slope Stability Analyses

6.5.1 We performed the slope stability analyses using the computer software program *GeoStudio 2004*, to calculate the factor of safety with respect to deep-seated instability. This program uses conventional slope stability equations and a two-dimensional, limit-equilibrium method. We performed the rotational-mode and block-mode analyses using Spencer's method. Output of the computer program including the calculated Factor of Safety and the failure surface is shown in Appendix C.

6.5.2 Our slope stability analyses utilized average-drained direct shear strength parameters based on laboratory test results and our experience with similar soil types in nearby areas. Our analyses indicate the proposed cut and fill slopes, constructed of on-site materials, should have calculated factors of safety of at least 1.5 under static conditions for both deep-seated failure and shallow sloughing conditions, if the recommendations of this report are followed. The shear strength parameters used in the slope stability analyses are summarized in Appendix C.

6.5.3 Cross-Sections B1-B1′, C-C′, G-G′, I-I′, and J-J′ are presented on Figures 9 through 11. We selected Cross-Sections B1-B1′, C-C′, and G-G′ as the most critical to perform the slope stability analyses. Proposed grading will expose the interbedded layers of sandstone and claystone of the Otay Formation and Cross-Sections B1-B1′, C1-C1′, and G-G′ appear to be the most critical cross-sections to be analyzed. A factor of safety of 1.5 for static conditions is currently required by the City of Chula Vista and the County of San Diego for permanent graded slopes. The slope stability results are presented in Appendix C. Table C-II in Appendix C provides a description of the cross-sections, their corresponding factor of safety, and the condition of the slope stability analyses.

6.5.4 We performed surficial slope stability calculations for a 2:1 (horizontal to vertical) slope in compacted fill. The calculated factor of safety is greater than the required minimum factor of safety of 1.5. Plants with variable root depth should be planted as soon as practical once cut and fill slopes have been constructed. Surficial slope stability calculations for compacted fill slopes are presented on Figure C-5 in Appendix C.

6.5.5 Excavations of cut slopes should be observed during grading by an engineering geologist to evaluate whether the soil and geologic conditions differ significantly from those expected. Cut slopes that expose sheared claystone bedding may require slope stabilization consisting of stability fills.

6.6 Grading

6.6.1 Grading should be performed in accordance with the *Recommended Grading Specifications* provided in Appendix D. If the recommendations of this section of the report conflict with Appendix D, the recommendations provided within this section take precedence.

6.6.2 Earthwork should be observed, and compacted fill tested by a representative of Geocon Incorporated. It is important that the firm that performed the geotechnical investigation for the project be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If Geocon Incorporated is not retained to provide these services, we would no longer be the Geotechnical Engineer of Record for the project, and must disclaim all responsibility for geotechnical aspects of the future performance of the project.

6.6.3 A preconstruction conference should be held at the site prior to the beginning of grading operations with representatives from the county/city, owner, contractor, civil engineer and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

6.6.4. Grading should commence with the removal of existing vegetation and underground improvements from the area of widening and realignment. Deleterious debris should be exported from the areas to be graded and should not be mixed with proposed fill. Existing underground improvements planned for removal should be removed and the resulting depressions properly backfilled in accordance with the procedures described herein. Existing CMP storm drains not planned to be removed during improvement should be filled with slurry, capped, and properly abandoned.

6.6.5 Undocumented fill and the underlying alluvium within the existing roadway prism are considered suitable to support the proposed retaining wall footings and pre-cast concrete animal under crossing structures. The surficial soil located within the canyon drainages in areas of roadway widening and realignment will require remedial grading consisting of removal and recompaction. The surficial soil should be removed to expose formational materials and replaced with properly compacted fill. The existing fill slopes in area of road widening will require significant benching during grading operations to remove erosion and the loose outer portion of the fill. Loose or saturated areas encountered during the grading operations may require removal and replacement with properly compacted fill.

6.6.6 In areas to receive fill subsequent to remedial grading, the exposed ground surface should be scarified to a depth of at least 12 inches, moisture conditioned as necessary, and properly compacted. Fill materials should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM Test Method D 1557.

6.6.7 Excavated soil that is free of deleterious debris can be placed as fill and compacted in layers to the design finish-grade elevations. Retaining wall backfill and structural fill should be placed and compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM Test Method D 1557. The upper 12 inches of fill beneath pavement should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content shortly before paving operations.

6.7 Foundation Recommendations

6.7.1 The recommendations provided herein are applicable for use in design of the pre-cast reinforced concrete structures planned for the project. We expect the foundations will be supported on properly compacted fill or medium dense alluvium currently located below the existing undocumented fill.

6.7.2 Foundations should be at least 24 inches wide and extend at least 12 inches below lowest adjacent grade.

6.7.3 The recommended allowable bearing capacity for foundations bearing in properly compacted fill or medium dense, undocumented fill or underlying alluvium is 2,000 pounds per square foot (psf). The allowable soil bearing pressure may be increased by an additional 500 psf for each additional foot of depth and 400 psf for each additional foot of width, to a maximum allowable bearing capacity of 3,500 psf. Total and differential settlement of structures imposing the maximum allowable bearing pressure is expected to be approximately 1 inch.

6.7.4 The values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

6.7.5 Foundation excavations should be observed by Geocon Incorporated prior to the placement of reinforcing steel and/or concrete structures to check that the exposed soil conditions are consistent with those expected. If unexpected soil conditions are encountered, foundation modifications may be required.

6.7.6 Footings located within 7 feet of the top of slopes are not recommended. However, footings that must be located within this zone should be extended in depth such that the outer bottom edge of the footing is at least 7 feet horizontally inside the face of the slope.

6.8 Retaining Wall Loads

6.8.1 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 35 pounds per cubic foot (pcf). Where the backfill will be inclined at 2:1 (horizontal:vertical), an active soil pressure of 50 pcf is recommended. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

6.8.2 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall. Where walls are restrained from movement at the top, an additional uniform pressure of 7H psf should be added to the active soil pressure. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge load equivalent to 2 feet of fill soil should be added.

6.8.3 The retaining wall foundations can be designed using the parameters presented in the *Foundation Recommendations* section of this report.

6.8.4 The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations presented herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. Figure 12 presents a typical retaining wall drain detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

6.8.5 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 18.3.5.12 of the 2013 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. A seismic load of 13H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.336g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

6.8.6 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

6.9 Lateral Loads

6.9.1 For resistance to lateral loads, an allowable passive earth pressure equivalent to a fluid density of 300 pcf is recommended for footings or shear keys poured neat against properly compacted granular fill or undisturbed formational materials. The allowable passive pressure assumes a horizontal surface extending away from the base of the wall at least 5 feet or three times the height of the surface generating the passive pressure, whichever is greater. The upper 12 inches of material not protected by pavement should not be included in the design for lateral resistance.

6.9.2 An allowable friction coefficient of 0.35 may be used for resistance to sliding between soil and concrete. This friction coefficient may be combined with the allowable passive earth pressure when determining resistance to lateral loads.

6.10 Mechanically Stabilized Earth (MSE) Retaining Walls

6.10.1 Mechanically stabilized earth (MSE) segmental retaining walls are alternative walls that consist of modular block facing units with geogrid-reinforced earth behind the block. The reinforcement grid attaches to the block units and is typically placed at specified vertical intervals and embedment lengths. For the purposes of this report, the spacing and lengths and types of the geogrid were assumed based on the expected type of soil used for the backfill, and the slope stability requirements to achieve an acceptable factor of safety. The onsite soil may contain oversize gravel and cobble material that typically requires screening prior to use as backfill material. Portions of the Otay Formation contain layers of clayey sandstone that will have less gravel and cobble. The siltstone and claystone portion of formational units is not considered acceptable materials for use as backfill.

6.10.2 The geotechnical parameters listed in Table 6.10 can be used for preliminary design of the MSE retaining walls.

TABLE 6.10
GEOTECHNICAL PARAMETERS FOR MSE WALLS

Parameter	Reinforced Zone	Retained Zone	Foundation Zone
Angle of Internal Friction	30 degrees	30 degrees	30 degrees
Cohesion	200 psf	200 psf	200 psf
Wet Unit Density	120 pcf	120 pcf	120 pcf

6.10.3 The soil parameters presented in Table 6.10 are based on our experience and direct shear-strength tests performed during the geotechnical investigation and represent some of the on-site materials. The wet unit density values presented in Table 6.10 can be used for design but actual in-place densities may range from approximately 90 to 135 pounds per cubic foot. Geocon has no way of knowing whether these materials will actually be used as backfill behind the wall during construction. It is up to the wall designers to use their judgment in selection of the design parameters. As such, once backfill materials have been selected and/or stockpiled, sufficient shear tests should be conducted on samples of the proposed backfill materials to check that they conform to actual design values. Results should be provided to the designer to re-evaluate stability of the walls. Dependent upon test results, the designer may require modifications to the original wall design (e.g., longer reinforcement embedment lengths and/or steel reinforcement).

6.10.4 The foundation zone is the area where the footing is embedded, the reinforced zone is the area of the backfill that possesses the reinforcing fabric, and the retained zone is the area behind the reinforced zone.

6.10.5 Wall foundations having a minimum depth and width of one foot may be designed for an allowable soil bearing pressure of 2,000 psf where the footings are founded on properly compacted fill or formational materials. This soil pressure may be increased by 300 psf for each additional foot of foundation width and depth up to a maximum allowable soil bearing pressure of 3,500 psf.

6.10.6 Backfill materials within the reinforced zone should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557. This is applicable to the entire embedment width of the reinforcement. Typically, wall designers specify no heavy compaction equipment within 3 feet of the face of the wall. However, smaller equipment (e.g., walk-behind, self-driven compactors or hand whackers) can be used to compact the materials without causing deformation of the wall. If the designer specifies no compactive effort for this zone, the materials are essentially not properly compacted and the reinforcement grid within the uncompacted zone should not be relied upon for reinforcement, and overall embedment lengths will have to be increased to account for the difference.

6.10.7 Select backfill materials may be required to be in accordance with the MSE retaining wall system. Materials as outlined in the specifications of the retaining wall plans may be generated and stockpiled during grading, if encountered, or may require import. Geocon Incorporated should perform laboratory tests during the backfill materials to check that soil properties are in accordance with the retaining wall plans and specifications.

6.10.8 The wall should be provided with a drainage system sufficient to prevent excessive seepage through the wall and the base of the wall, thus preventing hydrostatic pressures behind the wall.

6.10.9 Geosynthetic reinforcement must elongate to develop full tensile resistance. This elongation generally results in movement at the top of the wall. The amount of movement is dependent upon the height of the wall (e.g., higher walls rotate more) and the type of reinforcing grid used. In addition, over time the reinforcement grid has been known to exhibit creep (sometimes as much as 5 percent) and can undergo additional movement. Given this condition, the owner should be aware that structures and pavement placed within the reinforced and retained zones of the wall may undergo movement.

6.10.10 The MSE wall contractor should provide the estimated deformation of wall and adjacent ground in association with wall construction. The calculated horizontal and vertical deformations should be determined by the wall designer. The estimated movements should be provided to the project structural engineer to determine if the planned and existing improvements can tolerate the expected movements. In addition, a third-party structural plan review should be considered for the MSE walls subsequent to the completion of the plans.

6.11 Preliminary Pavement Recommendations

6.11.1 The final pavement sections for roadways should be based on the laboratory R-Value test results of the subgrade soils encountered at final subgrade elevation. Streets should be designed in accordance with the County of San Diego specifications when final Traffic Indices and R-value test results of subgrade soil are completed. Based on the results of our laboratory R-Value testing and our experience with similar soils in the area, we have assumed an R-Value of 15 for the subgrade soil for the purposes of this preliminary analysis. Preliminary flexible pavement sections are presented in Table 6.11.1. We have been provided a Traffic Index of 8.0 for the Major classified 4-lane portion of the proposed roadway and a Traffic Index of 7.0 for the Community Collector classified 2-lane portion.

TABLE 6.11.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Location	Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Community Collector	7.0	15	4	13
Major	8.0	15	5	15

6.11.2 The upper 12 inches of the subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content beneath pavement sections shortly before paving.

6.11.3 Base materials should conform to Section 26-1.028 of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* with a ¾-inch maximum size aggregate. Base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

6.11.4 If rigid Portland cement concrete (PCC) pavement is placed within the roadway the following rigid pavement recommendations should be used. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 6.11.2.

TABLE 6.11.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	500 psi
Traffic Category, TC	D
Average daily truck traffic, ADTT	700

6.11.5 Based on the criteria presented herein, the PCC pavement section should have a minimum thickness as presented in Table 6.11.3.

TABLE 6.11.3
PRELIMINARY RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Major and Community Collector	8

6.11.6 The PCC pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content shortly before paving. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch).

6.11.7 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, at the slab edge and taper back to the recommended slab thickness 3 feet behind the face of the slab (e.g., 8-inch-thick slab would have a 10-inch-thick edge).

6.11.8 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 15 feet (e.g., 8-inch-thick slab would have a 16-foot spacing pattern) and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report.

6.11.9 To provide load transfer between adjacent pavement slab sections, a trapezoidal-keyed construction joint is recommended. As an alternative to the keyed joint, dowelling is recommended between construction joints. As discussed in the referenced ACI guide, dowels should consist of smooth, ⅞-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. Other alternative recommendations for load transfer should be provided by the project structural engineer.

6.11.10 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab. As an alternative to the butt-type construction joint, dowelling can be used between construction joints for pavements of 7 inches or thicker. As discussed in the referenced ACI guide, dowels should consist of smooth, 1-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. In addition, tie bars should be installed at the as recommended in Section 3.8.3 of the referenced ACI guide. The structural engineer should provide other alternative recommendations for load transfer.

6.12 Site Drainage and Moisture Protection

6.12.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond on or adjacent to pavement surfaces. The site should be graded and maintained such that surface drainage is directed away from pavement areas in accordance with 2013 CBC 1804.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices.

6.12.2 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

6.12.3 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material. Cutoff walls would not be required where hardscape in installed within the area of the median or where zero-maintenance landscape (without the installation of irrigation lines) is installed.

6.12.4 If detention basins, bioswales, retention basins, or water infiltration devices are being considered, Geocon Incorporated should be retained to provide recommendations pertaining to the geotechnical aspects of possible impacts and design. Distress may be caused to planned improvements and properties located hydrologically downstream. The distress depends on the amount of water to be detained, its residence time, soil permeability, and other factors. We have not performed a hydrogeology study at the site. Downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other impacts as a result of water infiltration.

6.13 Grading Plan Review

6.13.1 Geocon Incorporated should review the grading plans prior to finalization to check their compliance with the recommendations of this report and determine the necessity for additional comments, recommendations and/or analyses.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate 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, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record

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VICINITY MAP

OTAY RANCH RESORT VILLAGE
OTAY LAKES ROAD WIDENING AND REALIGNMENT
SAN DIEGO COUNTY, CALIFORNIA

DATE 11 - 11 - 2014	PROJECT NO. G1012 - 52 - 01A	FIG. 1

Ladrillono

Y:\PROJECTS\G1012-52-01A OTAY LAKES RD WIDENING (Upd 11-03-2014)\DETAILS\G1012-52-01A Vic Map.dwg

SEE FIGURE 3

SEE FIGURE 3

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL
Qal ....ALLUVIUM
Qoal ....OLDER ALLUVIUM
Ql ....LACUSTRINE DEPOSITS
Tof ........FANGLOMERATE DEPOSITS
To ....OTAY FORMATION
KJmv ........METAVOLCANIC ROCK
........APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
ET-9 ....APPROX. LOCATION OF EXCAVATOR TRENCH
NT-3 ....APPROX. LOCATION OF PREVIOUS BACKHOE TRENCH (Neblett & Associates, 2004)
AT-1 ....APPROX. LOCATION OF AIR TRACK BORING
LB-4 ....APPROX. LOCATION OF LARGE DIAMETER BORING
SB-10 ....APPROX. LOCATION OF SMALL DIAMETER BORING
T-22 ....APPROX. LOCATION OF TRACKHOE TRENCH
J J' ....APPROX. LOCATION OF GEOLOGIC CROSS-SECTION
........APPROX. LOCATION OF PROPOSED SUBDRAIN
1.5 ....ESTIMATED REMOVAL DEPTH FOR REMEDIAL GRADING (In Feet)
2 ....ESTIMATED THICKNESS OF CAPPING MATERIAL AFTER SCREENING (In Feet)
>13 ....ESTIMATED RIPPABLE DEPTH DURING GRADING (In Feet)

GEOLOGIC MAP

OTAY RANCH RESORT VILLAGE
OTAY LAKES ROAD WIDENING AND REALIGNMENT
SAN DIEGO COUNTY, CALIFORNIA

GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

SCALE 1" = 100'	DATE 11 - 11 - 2014
PROJECT NO. G1012 - 52 - 01A	FIGURE 2
SHEET 1 OF 7	

Plotted 11/11/2014 10:55AM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G1012-52-01A OTAY LAKES RD WIDENING (Upd 11-03-2014)\SHEETS\G1012-52-01A_OTR-Widening GeoMap (Upd 11-03-2014).dwg

SEE FIGURE 4

GEOCON LEGEND

- **Qudf** ........UNDOCUMENTED FILL
- **Qal** ....ALLUVIUM
- **Qoal** ....OLDER ALLUVIUM
- **Ql** ....LACUSTRINE DEPOSITS
- **Tof** ........FANGLOMERATE DEPOSITS
- **To** ....OTAY FORMATION
- **KJmv** ........METAVOLCANIC ROCK
- ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
- ET-9 ....APPROX. LOCATION OF EXCAVATOR TRENCH
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- 1.5 ....ESTIMATED REMOVAL DEPTH FOR REMEDIAL GRADING (In Feet)
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- >13 ....ESTIMATED RIPPABLE DEPTH DURING GRADING (In Feet)

GEOLOGIC MAP

OTAY RANCH RESORT VILLAGE
OTAY LAKES ROAD WIDENING AND REALIGNMENT
SAN DIEGO COUNTY, CALIFORNIA

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SCALE 1" = 100'	DATE 11 - 11 - 2014	
PROJECT NO. G1012 - 52 - 01A		FIGURE 3
SHEET 2 OF 7		

SEE FIGURE 2

GEOCON LEGEND

Qudf	UNDOCUMENTED FILL
Qal	ALLUVIUM
Qoal	OLDER ALLUVIUM
Ql	LACUSTRINE DEPOSITS
Tof	FANGLOMERATE DEPOSITS
To	OTAY FORMATION
KJmv	METAVOLCANIC ROCK
?	APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
ET-9	APPROX. LOCATION OF EXCAVATOR TRENCH
NT-3	APPROX. LOCATION OF PREVIOUS BACKHOE TRENCH (Neblett & Associates, 2004)
AT-1	APPROX. LOCATION OF AIR TRACK BORING
LB-4	APPROX. LOCATION OF LARGE DIAMETER BORING
SB-10	APPROX. LOCATION OF SMALL DIAMETER BORING
T-22	APPROX. LOCATION OF TRACKHOE TRENCH
J J'	APPROX. LOCATION OF GEOLOGIC CROSS-SECTION
	APPROX. LOCATION OF PROPOSED SUBDRAIN
1.5	ESTIMATED REMOVAL DEPTH FOR REMEDIAL GRADING (In Feet)
2	ESTIMATED THICKNESS OF CAPPING MATERIAL AFTER SCREENING (In Feet)
>13	ESTIMATED RIPPABLE DEPTH DURING GRADING (In Feet)

GEOLOGIC MAP

OTAY RANCH RESORT VILLAGE
OTAY LAKES ROAD WIDENING AND REALIGNMENT
SAN DIEGO COUNTY, CALIFORNIA

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SCALE 1" = 100'	DATE 11 - 11 - 2014	
PROJECT NO. G1012 - 52 - 01A		FIGURE 4
SHEET 3 OF 7		

2014 10:43AM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G1012-52-01A OTAY LAKES RD WIDENING (Upd 11-03-2014)\SHEETS\G1012-52-01A_OTR-Widening GeoMap (Upd 11-03-2014).dwg

SEE FIGURE 4

SEE FIGURE 6

GEOCON LEGEND

- **Qudf** ........UNDOCUMENTED FILL
- **Qal** ....ALLUVIUM
- **Qoal** ....OLDER ALLUVIUM
- **Ql** ....LACUSTRINE DEPOSITS
- **Tof** ........FANGLOMERATE DEPOSITS
- **To** ....OTAY FORMATION
- **KJmv** ........METAVOLCANIC ROCK
- ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
- ET-9 ....APPROX. LOCATION OF EXCAVATOR TRENCH
- NT-3 ....APPROX. LOCATION OF PREVIOUS BACKHOE TRENCH (Neblett & Associates, 2004)
- AT-1 ....APPROX. LOCATION OF AIR TRACK BORING
- LB-4 ....APPROX. LOCATION OF LARGE DIAMETER BORING
- SB-10 ....APPROX. LOCATION OF SMALL DIAMETER BORING
- T-22 ....APPROX. LOCATION OF TRACKHOE TRENCH
- J J' ....APPROX. LOCATION OF GEOLOGIC CROSS-SECTION
- ....APPROX. LOCATION OF PROPOSED SUBDRAIN
- 1.5 ....ESTIMATED REMOVAL DEPTH FOR REMEDIAL GRADING (In Feet)
- 2 ....ESTIMATED THICKNESS OF CAPPING MATERIAL AFTER SCREENING (In Feet)
- >13 ....ESTIMATED RIPPABLE DEPTH DURING GRADING (In Feet)

GRAPHIC SCALE

0' 50' 100' 150' 200' 300' 400'

SCALE 1" = 100'

GEOLOGIC MAP

OTAY RANCH RESORT VILLAGE
OTAY LAKES ROAD WIDENING AND REALIGNMENT
SAN DIEGO COUNTY, CALIFORNIA

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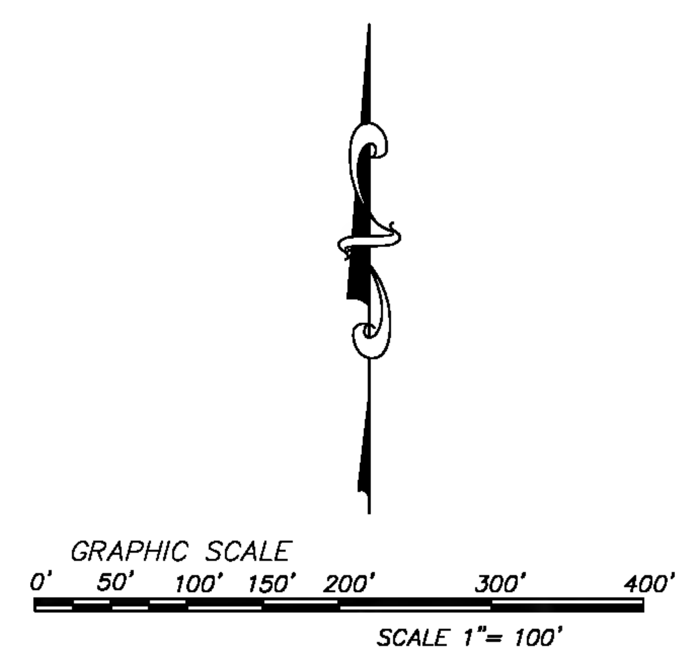
SCALE 1" = 100'	DATE 11 - 11 - 2014
PROJECT NO. G1012 - 52 - 01A	FIGURE 5
SHEET 4 OF 7	

SEE FIGURE 5

SEE FIGURE 7



GEOCON LEGEND

- **Qudf** ........UNDOCUMENTED FILL
- **Qal** ....ALLUVIUM
- **Qoal** ....OLDER ALLUVIUM
- **Ql** ....LACUSTRINE DEPOSITS
- **Tof** ........FANGLOMERATE DEPOSITS
- **To** ....OTAY FORMATION
- **KJmv** ........METAVOLCANIC ROCK
- ? ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
- ET-9 ....APPROX. LOCATION OF EXCAVATOR TRENCH
- NT-3 ....APPROX. LOCATION OF PREVIOUS BACKHOE TRENCH (Neblett & Associates, 2004)
- AT-1 ....APPROX. LOCATION OF AIR TRACK BORING
- LB-4 ....APPROX. LOCATION OF LARGE DIAMETER BORING
- SB-10 ....APPROX. LOCATION OF SMALL DIAMETER BORING
- T-22 ....APPROX. LOCATION OF TRACKHOE TRENCH
- J J' ....APPROX. LOCATION OF GEOLOGIC CROSS-SECTION
- ....APPROX. LOCATION OF PROPOSED SUBDRAIN
- 1.5 ....ESTIMATED REMOVAL DEPTH FOR REMEDIAL GRADING (In Feet)
- 2 ....ESTIMATED THICKNESS OF CAPPING MATERIAL AFTER SCREENING (In Feet)
- >13 ....ESTIMATED RIPPABLE DEPTH DURING GRADING (In Feet)

GEOLOGIC MAP

OTAY RANCH RESORT VILLAGE
OTAY LAKES ROAD WIDENING AND REALIGNMENT
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SCALE 1" = 100'	DATE 11 - 11 - 2014	
PROJECT NO. G1012 - 52 - 01A		FIGURE 6
SHEET 5 OF 7		

By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G1012-52-01A OTAY LAKES RD WIDENING (Upd 11-03-2014)\SHEETS\G1012-52-01A_OTR-Widening GeoMap (Upd 11-03-2014).dwg

GEOLOGIC MAP

OTAY RANCH RESORT VILLAGE
OTAY LAKES ROAD WIDENING AND REALIGNMENT
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SCALE 1" = 100'	DATE 11 - 11 - 2014
PROJECT NO. G1012 - 52 - 01A	FIGURE 7
SHEET 6 OF 7	


SEE FIGURE 7
KJmv
Qal
Qoal
Qudf
Tof
To
OS 13B-A
OPEN SPACE
HOA LOT 13B-35
HOA LOT 13B-34
OTAY LAKES ROAD
EXISTING PROJECT BOUNDARY
PROPOSED PROJECT BOUNDARY
PROJECT BOUNDARY
AREA OF ROCK FALL MITIGATION
EXTENDS 2,540 FEET
EXTENDS 1,420 FEET
SUBDRAIN OUTLET
CONNECT TO EXISTING TRAIL SYSTEM
PROPOSED GRADING DAYLIGHT LINE
PROPOSED BROW DITCH
PROPOSED DRAINAGE BROW DITCH
EXISTING AIRPORT DRIVEWAY ACCESS
T-23
T-26
T-28
T-29
T-30
T-31
ET-34
ET-48
SB-10
GEOCON LEGEND
Qudf ......UNDOCUMENTED FILL
Qal ......ALLUVIUM
Qoal ......OLDER ALLUVIUM
Ql ......LACUSTRINE DEPOSITS
Tof ......FANGLOMERATE DEPOSITS
To ......OTAY FORMATION
KJmv ......METAVOLCANIC ROCK
......APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
ET-8 ......APPROX. LOCATION OF EXCAVATOR TRENCH
NT-3 ......APPROX. LOCATION OF PREVIOUS BACKHOE TRENCH (Nebleit & Associates, 2004)
AT-1 ......APPROX. LOCATION OF AIR TRACK BORING
LB-4 ......APPROX. LOCATION OF LARGE DIAMETER BORING
SB-10 ......APPROX. LOCATION OF SMALL DIAMETER BORING
T-22 ......APPROX. LOCATION OF TRACKHOE TRENCH
......APPROX. LOCATION OF GEOLOGIC CROSS-SECTION
......APPROX. LOCATION OF PROPOSED SUBDRAIN
1.5 ......ESTIMATED REMOVAL DEPTH FOR REMEDIAL GRADING (In Feet)
2 ......ESTIMATED THICKNESS OF CAPPING MATERIAL AFTER SCREENING (In Feet)
>13 ......ESTIMATED RIPPABLE DEPTH DURING GRADING (In Feet)
GRAPHIC SCALE
0' 50' 100' 150' 200' 300' 400'
SCALE 1" = 100'
GEOLOGIC MAP
OTAY RANCH RESORT VILLAGE
OTAY LAKES ROAD WIDENING AND REALIGNMENT
SAN DIEGO COUNTY, CALIFORNIA
GEOCON INCORPORATED
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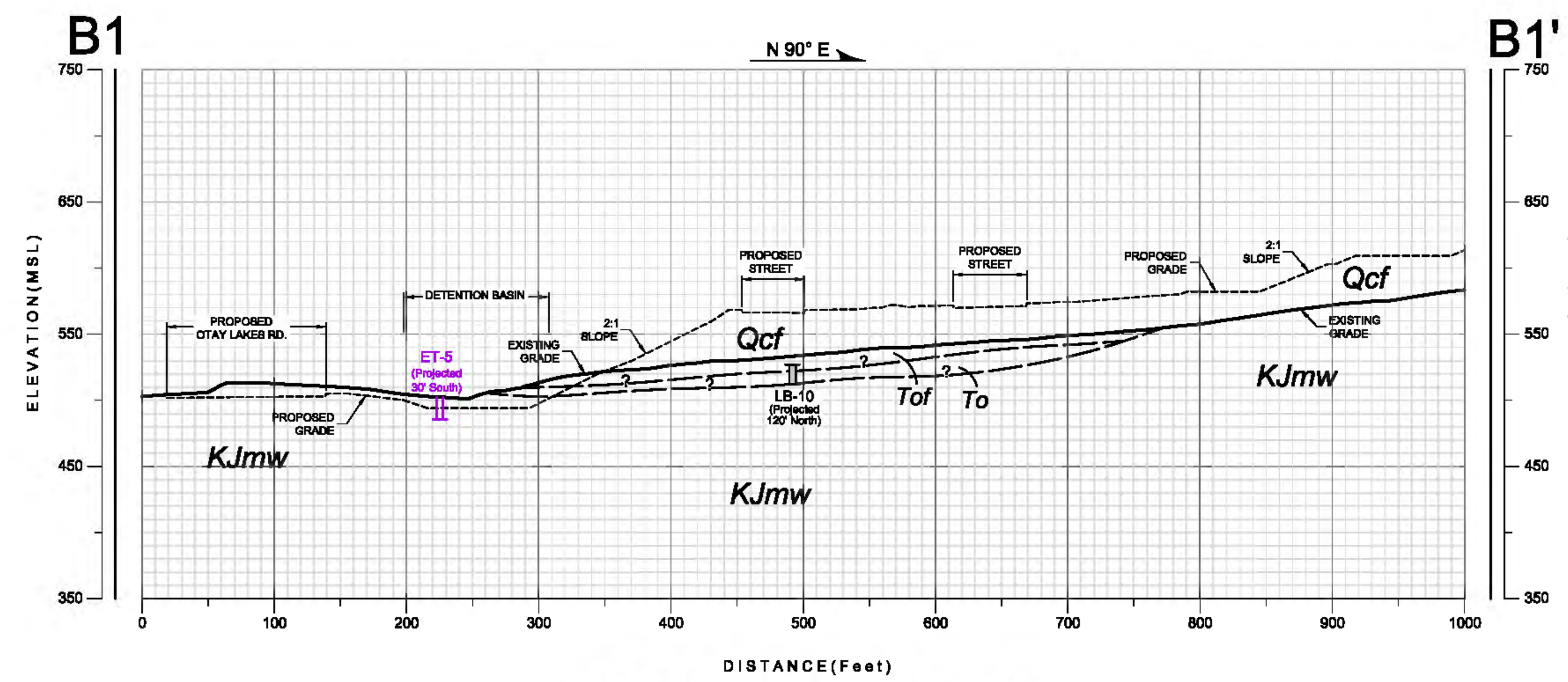
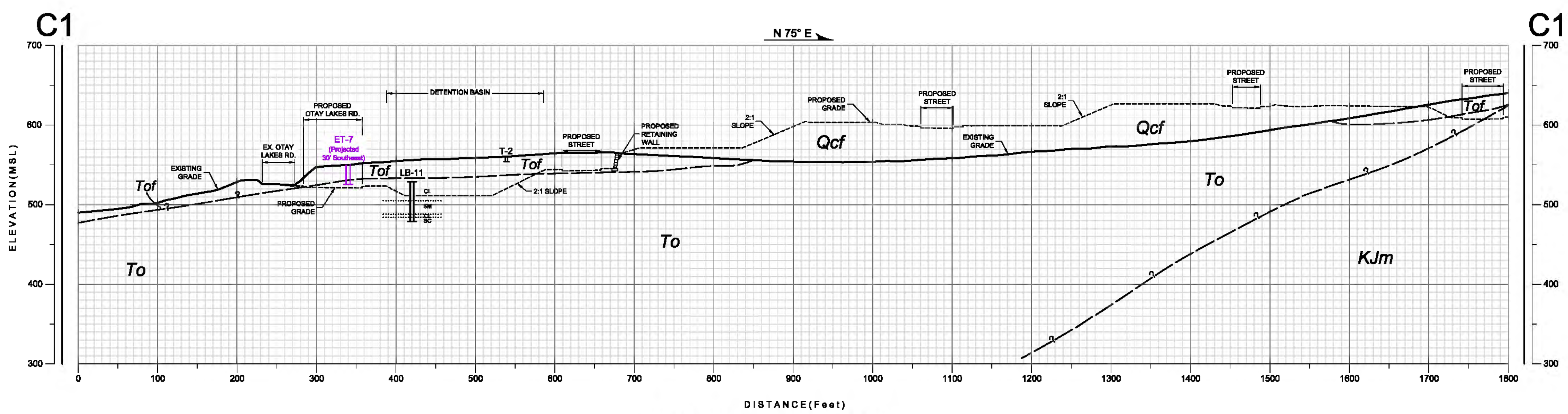
SCALE 1" = 100'
DATE 11 - 11 - 2014
PROJECT NO. G1012 - 52 - 01A
FIGURE 8
SHEET 7 OF 7



GEOLOGIC CROSS-SECTION B-B'

SCALE: 1" = 100' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION C1-C1'

SCALE: 1" = 100' (Vert. = Horiz.)

GEOCON LEGEND

- Qcf ....COMPACTED FILL (Proposed)
- Qudf ........UNDOCUMENTED FILL
- Ql ....LACUSTRINE DEPOSITS
- Qal ....ALLUVIAL DEPOSITS
- Qt ....TERRACE DEPOSITS
- Tof ....TERTIARY-AGE FANGLOMERATE
- To ....OTAY FORMATION
- KJmv ........METAVOLCANIC BEDROCK
- ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
- AT-22 ....APPROX. LOCATION OF AIR TRACK BORING
- AH-10 ....APPROX. LOCATION OF PREVIOUS AIR TRACK BORING (Neblett & Associates, 2004)
- ET-48 ....APPROX. LOCATION OF EXCAVATOR TRENCH
- NT-47 ....APPROX. LOCATION OF PREVIOUS BACKHOE TRENCH (Neblett & Associates, 2004)
- LB-12 ....APPROX. LOCATION OF LARGE DIAMETER BORING
- SB-7 ....APPROX. LOCATION OF SMALL DIAMETER BORING
- T-9 ....APPROX. LOCATION OF TRENCH

GEOLOGIC CROSS - SECTIONS

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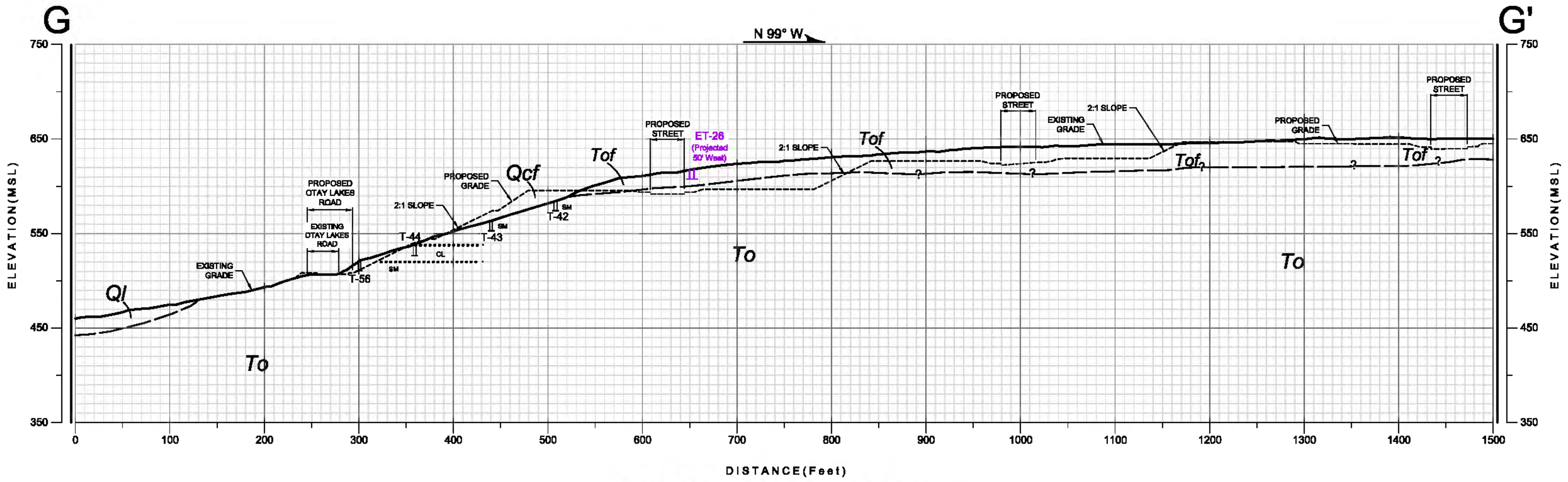
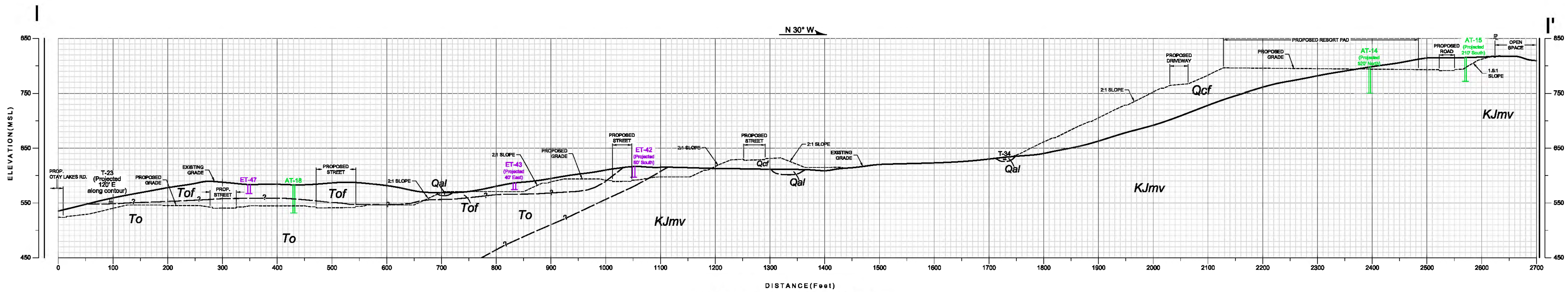
SCALE 1" = 100'	DATE 11 - 11 - 2014
PROJECT NO. G1012 - 52 - 01A	FIGURE 9
SHEET 1 OF 3	



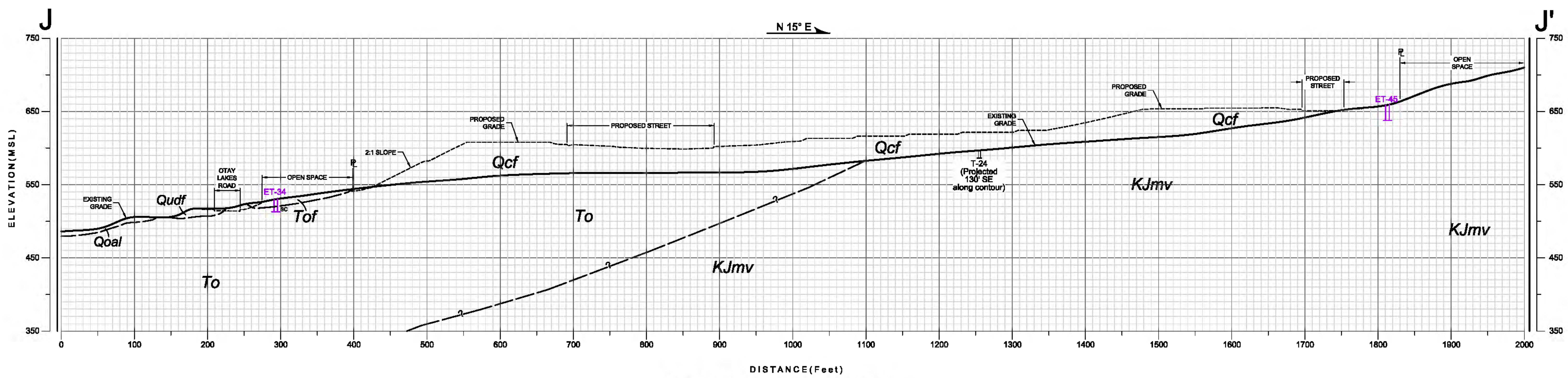
GEOLOGIC CROSS-SECTION G-G'

SCALE: 1" = 100' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION I-I'

SCALE: 1" = 100' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION J-J'

GEOCON LEGEND

- Qcf ....COMPACTED FILL (Proposed)
- Qudf ........UNDOCUMENTED FILL
- Ql ....LACUSTRINE DEPOSITS
- Qal ....ALLUVIAL DEPOSITS
- Qt ....TERRACE DEPOSITS
- Tof ....TERTIARY-AGE FANGLOMERATE
- To ....OTAY FORMATION
- KJmv ........METAVOLCANIC BEDROCK
- ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
- AT-22 ....APPROX. LOCATION OF AIR TRACK BORING
- AH-10 ....APPROX. LOCATION OF PREVIOUS AIR TRACK BORING (Neblett & Associates, 2004)
- ET-48 ....APPROX. LOCATION OF EXCAVATOR TRENCH
- NT-47 ....APPROX. LOCATION OF PREVIOUS BACKHOE TRENCH (Neblett & Associates, 2004)
- LB-12 ....APPROX. LOCATION OF LARGE DIAMETER BORING
- SB-7 ....APPROX. LOCATION OF SMALL DIAMETER BORING
- T-8 ....APPROX. LOCATION OF TRENCH

GEOLOGIC CROSS - SECTIONS

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SCALE 1" = 100'	DATE 11 - 11 - 2014
PROJECT NO. G1012 - 52 - 01A	FIGURE 10
SHEET 2 OF 3	

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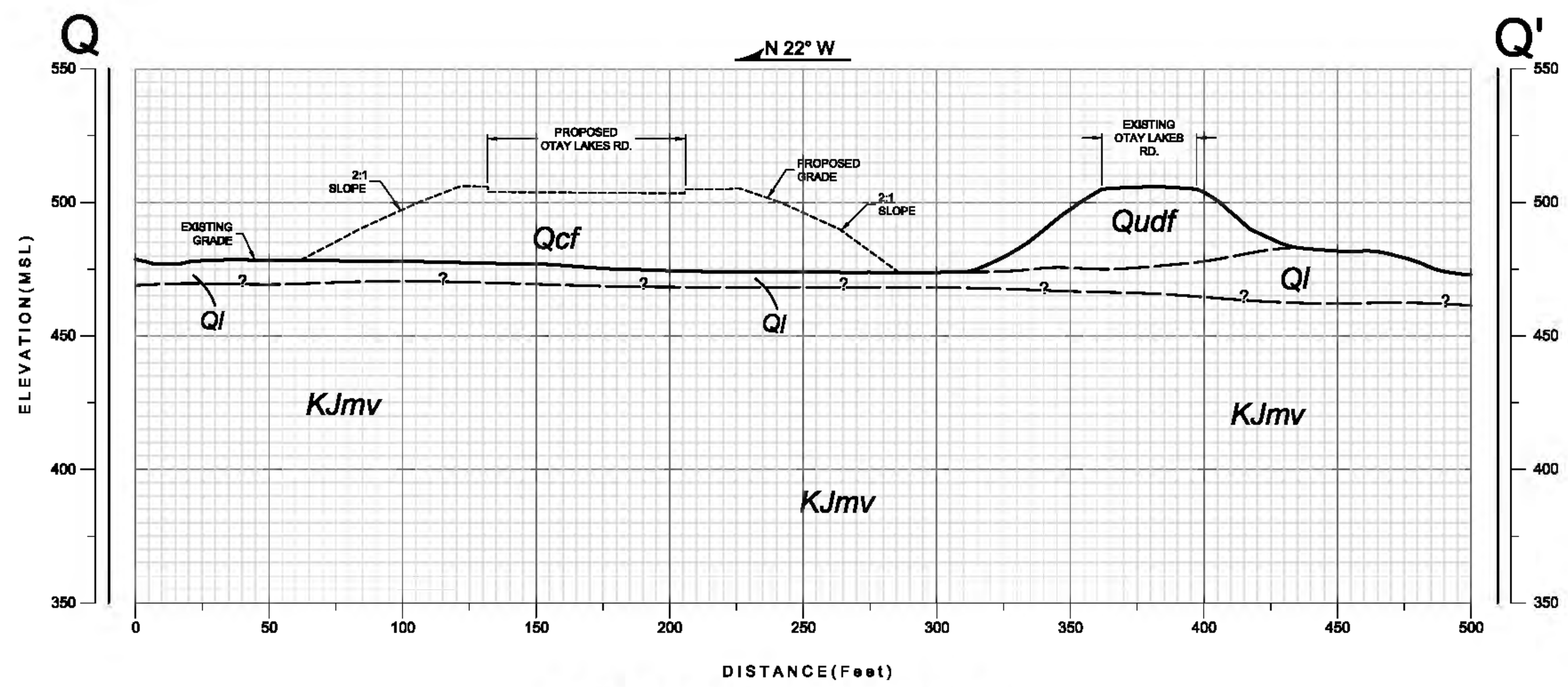
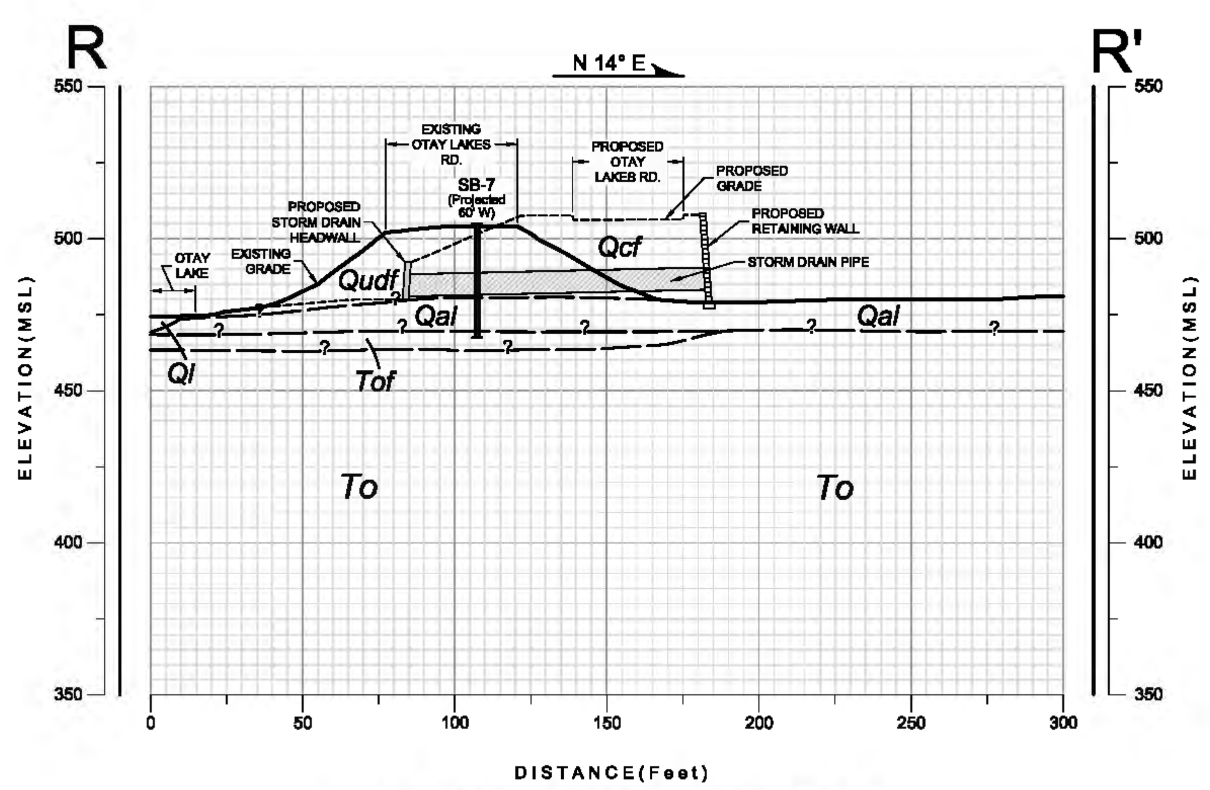


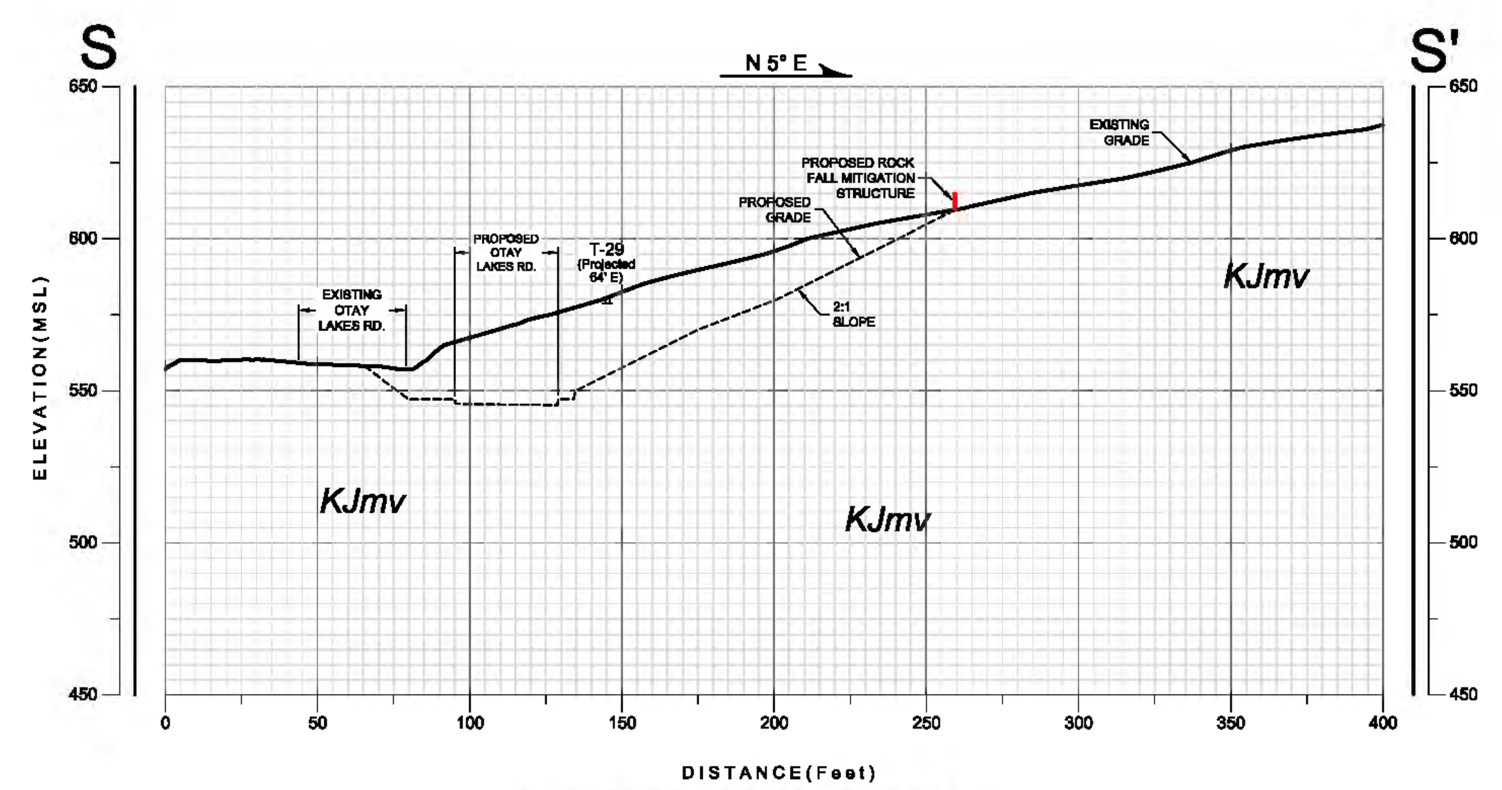
GEOLOGIC CROSS-SECTION Q-Q'

SCALE: 1" = 50' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION R-R'

SCALE: 1" = 50' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION S-S'

SCALE: 1" = 50' (Vert. = Horiz.)

GEOCON LEGEND

- *Qcf* ....COMPACTED FILL (Proposed)
- *Qudf* ........UNDOCUMENTED FILL
- *Ql* ....LACUSTRINE DEPOSITS
- *Qal* ....ALLUVIAL DEPOSITS
- *Qt* ....TERRACE DEPOSITS
- *Tof* ....TERTIARY-AGE FANGLOMERATE
- *To* ....OTAY FORMATION
- *KJmv* ........METAVOLCANIC BEDROCK
- ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
- AT-22 ....APPROX. LOCATION OF AIR TRACK BORING
- AH-10 ....APPROX. LOCATION OF PREVIOUS AIR TRACK BORING (Neblett & Associates, 2004)
- ET-48 ....APPROX. LOCATION OF EXCAVATOR TRENCH
- NT-47 ....APPROX. LOCATION OF PREVIOUS BACKHOE TRENCH (Neblett & Associates, 2004)
- LB-12 ....APPROX. LOCATION OF LARGE DIAMETER BORING
- SB-7 ....APPROX. LOCATION OF SMALL DIAMETER BORING
- T-9 ....APPROX. LOCATION OF TRENCH

GEOLOGIC CROSS - SECTIONS

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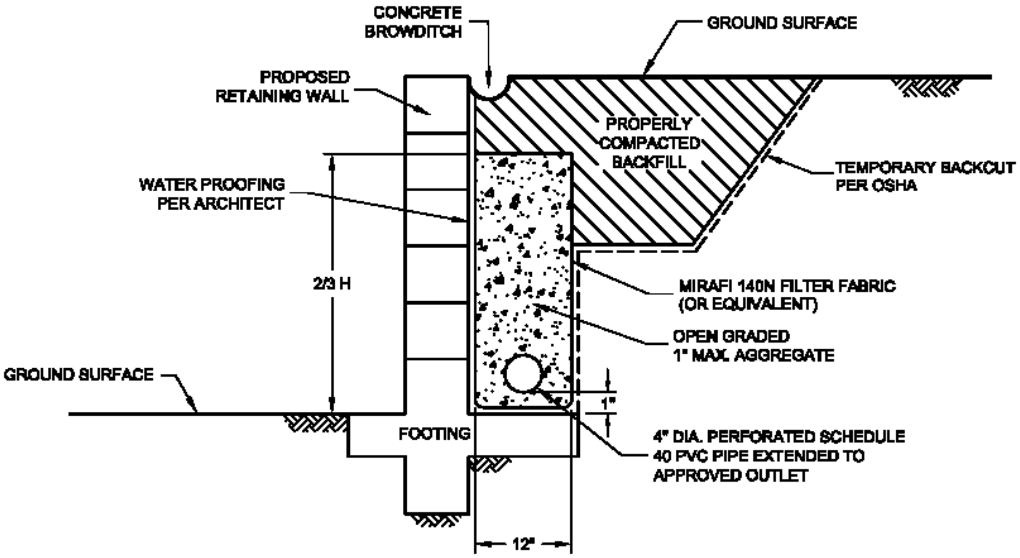
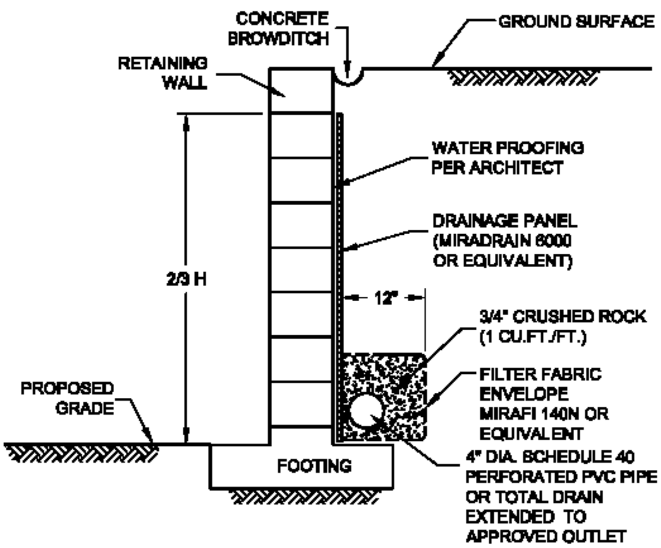
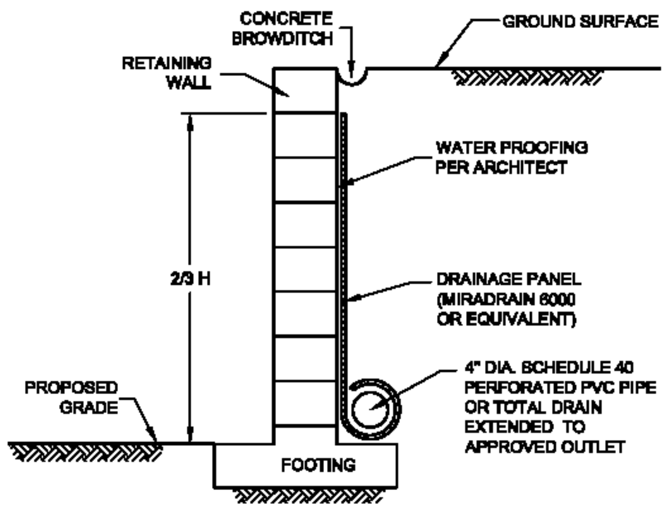
SCALE 1" = 50'	DATE 11 - 11 - 2014	
PROJECT NO. G1012 - 52 - 01A		FIGURE 11
SHEET 3 OF 3		

LADRILLONO







NOTE :

DRAIN SHOULD BE UNIFORMLY SLOPED TO GRAVITY OUTLET
OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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OTAY RANCH RESORT VILLAGE
OTAY LAKES ROAD WIDENING AND REALIGNMENT
SAN DIEGO COUNTY, CALIFORNIA

LR / AML		DSK/GTYPD	DATE 11 - 11 - 2014	PROJECT NO. G1012 - 52 - 01A	FIG. 12

RWDD7A

Y:\PROJECTS\G1012-52-01A OTAY LAKES RD WIDENING (Upd 11-03-2014)\DETAILS\RWDD7A.dwg

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