UPDATED
GEOTECHNICAL INVESTIGATION

EAST OTAY MESA CENTER MIXED-USE
OTAY MESA AND HARVEST ROADS
SAN DIEGO COUNTY, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

SUNROAD ENTERPRISES
SAN DIEGO, CALIFORNIA

JULY 20, 2015
PROJECT 06263-42-03

Project No. 06263-42-03
July 20, 2015

Sunroad Enterprises
4445 Eastgate Mall, Suite 400
San Diego, California 92121

Attention: Mrs. Andrea Contreras Rosati, Vice President and Counsel

Subject: UPDATED GEOTECHNICAL INVESTIGATION
EAST OTAY MESA CENTER MIXED-USE
OTAY MESA AND HARVEST ROADS
SAN DIEGO COUNTY, CALIFORNIA

Dear Mrs. Contreas:

In accordance with your authorization of our proposal (LG-15194 dated June 12, 2015), we herein submit the results of our Updated Geotechnical Investigation for the subject site. The accompanying report presents the findings and conclusions from our study. Based on the results of our study, it is our opinion that the subject site can be developed as proposed, provided the recommendations of this report are followed.

This updated report presents recommendations that should be incorporated into the phases of design and construction. The new recommendations supersede those presented in our reports titled *Soils and Geologic Investigation for Rancon Otay Mesa, dated November 15, 1990* and *Soil and Geologic Investigation for Sunroad Centrum (Rancon Otay Mesa)*, dated February 26, 1999. Differences between the recommendations are attributable to changes in the standard of geotechnical practice that have occurred since the issuing our previous reports. The recommendations presented herein are based on proposed grades shown on the project Preliminary Grading Plan.

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

Very truly yours,

GEOCON INCORPORATED

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UPDATED GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of an Updated Geotechnical Investigation for East Otay Mesa Center Mixed-Use project located in the Otay Mesa area of San Diego County, California (see Vicinity Map, Figure 1). The purpose of our work was to review our reports titled *Soil and Geologic Investigation for Rancon Otay Mesa* dated November 15, 1990, and *Updated Geotechnical Investigation for Sunroad Centrum (Rancon Otay Mesa),* dated February 26, 1999, and, based upon our review, to provide updated geotechnical recommendations pertaining to development of the property as presently proposed.

The scope of our services included the following:

- Reviewing our previous geotechnical investigation reports;
- Reviewing readily available published and unpublished geologic geotechnical reports pertaining to the area as indicated in the *List of References*.
- Performing a reconnaissance of the site;
- Plotting the exploratory borings and trenches on the preliminary grading plan;
- Producing four, geologic, cross-sections based on the soil conditions encountered in the exploratory borings and trenches;
- Preparing a geologic map over the preliminary grading plan;
- Reviewing existing grading, foundation and retaining wall recommendations;
- Preparing an updated geotechnical investigation report with updated grading and foundation recommendations based on the proposed grades presented on the preliminary grading plan.

The Geologic Map (Figure 2) was prepared using the *Preliminary Grading Plan* prepared by Stevens Cresto Engineering, plot date May 10, 2015. Stereoscopic aerial photographs dated 1953 (USDA, AXN-3M-24 and AXN-3M-25) were also analyzed to aid geologic mapping and identification of potential geologic constraints.

Laboratory tests were performed on selected representative soil samples obtained from the exploratory borings and trenches to evaluate pertinent physical properties. Descriptions of the field and laboratory procedures and methods are presented in Appendices A and B, respectively.

The conclusions and recommendations presented herein are based on analysis of the data obtained from our reviews, analysis of the laboratory test results, and our experience with similar soil and geologic conditions.

2. SITE AND PROJECT DESCRIPTION

The subject property encompasses approximately 250 acres of undeveloped land east and west of Harvest Road and immediately north of Otay Mesa Road in San Diego County, California (see Vicinity Map Figure 1 and Geologic Map, Figure 2, map pocket).

The property is nearly flat-lying to steeply sloping with elevations ranging from approximately 620 feet Mean Sea Level (MSL) in the central portion of the site to approximately 527 feet MSL at the northwest corner.

Existing improvements consist of Harvest Road at the west end, a dirt road along the east property line, several dirt roads trending east-west in the central portion of the site over the existing knoll, an abandoned borrow pit in the north-central portion, and several buried and surface irrigation lines. A seepage pit was observed at the southeast end of the site. Natural drainage is mainly a network of shallow swales and ravines that discharge into Johnson Canyon to the northeast (area designated as open space easement) or into controlled facilities along Otay Mesa Road to the south. Vegetation primarily consists of grasses with brush on the steeper slopes. The central-north section of the site is covered with an extensive volume of dumped soils, trash, and debris.

We understand that project will consist of grading the property to receive 29, sheet-graded lots of the East Otay Mesa Mixed-Use development with four major arterial streets and four interior streets. Improvements along Harvest Road and the widening of Otay Mesa Road along the frontage of the property are also planned. The area north of the proposed Lone Star Road is designated open space easement.

Review of the project preliminary grading plan indicates that cuts and fills on the order of 35 and 30 feet, respectively, are proposed to achieve subgrade elevations on the proposed industrial sheet-graded lots. We expect that the lots will be fine-graded at a later date on an individual basis. In addition, extensive remedial grading in the form of removal and compaction of existing topsoils, alluvium/colluvium and the weathered soil of the Otay Formation should be anticipated.

The buildings will be for industrial and/or commercial mixed use and will likely consist of concrete tilt-up walls with concrete reinforced, steel, and/or wood-frame structures, supported on conventional continuous and/or spread footings.

3. SOIL AND GEOLOGIC CONDITIONS

During our field investigation we encountered undocumented fill soil, topsoil, alluvium/colluvium, Old Terrace Deposits, and the Otay Formation. These units are described below.

3.1 Undocumented Fill Soils (Qudf)

Undocumented fill soils were observed throughout the north-central portion of the site. The undocumented fill soils contain considerable amounts of vegetation and debris. These soils should be cleaned of vegetation and any deleterious debris prior to being used as structural fill. We expect that that majority of this soil will be removed as part of the normal grading operations to achieve proposed grades.

3.2 Topsoil (Unmapped)

Soft clayey topsoil overlies the majority of the site and have a somewhat uniform thickness of 2 to 3 feet. The topsoil generally consists of silty to sandy clays and clayey sands. The topsoil is potentially compressible and/or highly expansive and will require remedial grading measures in the form of removal and compaction as indicated in the grading section.

3.3 Alluvium/Colluvium (Qal/Qc)

Undifferentiated alluvial/colluvial soils are composed primarily of compressible silty and sandy clays. The thickness of these soils range from 3 to 7 feet with an average of 5 feet. The alluvial/colluvial soils are unsuitable for the support of settlement-sensitive structures or structural fill soils. Accordingly, remedial grading will be required.

3.4 Old Terrace Deposits (Qt)

Quaternary-age Old Terrace Deposits consist of very dense, weakly-cemented to cohesionless sand, cobble, and boulders that cap the broad knoll in the central portion of the property and the southwestern corner of the site. Metavolcanic rock clasts are abundant and indicate that the Old Terrace Deposits probably originated from the nearby Otay Mountains. The soils of these deposits possess satisfactory foundation engineering characteristics in both undisturbed and properly compacted states. The presence of very large boulders (some in excess of 3 feet in diameter), as encountered in Trenches T1 through T6, is not uncommon and, if encountered during grading, may require special handling and placement techniques in compacted fills. Oversize rocks should be placed in accordance with Section 6.3 at Appendix C.

3.5 Otay Formation (To)

The Oligocene-age Otay Formation consists of very dense, light gray-brown to light brown, silty to clayey sandstones and hard, sandy claystones and siltstones. The sandy and clayey units vary in thickness and are typically interbedded. The sandier portions of the Otay Formation are considered to have *low* to *medium* expansive potential, whereas the clayey portions are *medium* to *high* in expansive potential. One bentonite clay seam, with critically *high* expansive potential also was encountered in the exploratory boring LB-7. The claystone units of the Otay Formation typically exhibit low shear strength and accordingly, landslides or other types of slope instability can occur where these soils are present. A study of the previously-referenced aerial photographs and geologic observations made during the drilling and trenching operations did not reveal the presence of landslides; however, we recommend that the potential impact of the Otay Formation on slope stability be further evaluated after final grading plans become available for review. Based on the preliminary grading plan, we expect that highly expansive bentonitic clays may be exposed within 10 feet of subgrade elevation in Lot 17 as indicated in Geologic Cross-section B-B', Figure 3. Highly weathered Otay Formation that requires remedial grading may be encountered where exposed at the surface or beneath alluvium/colluvium. Weathering extends to 5 to 8 feet in some locations.

4. GEOLOGIC STRUCTURE

The general geologic structure is a gently, southwesterly dipping planar strata. Data obtained from Borings B-1, B-2, and B-3 suggest that the Otay Formation generally strikes N60°W and dips 3°SW.

We observed remolded clay seams and/or fractured claystone within bentonitic layers within the Otay Formation during our subsurface investigation. These features are interpreted as bedding parallel shears and may be related to stress relief along weak beds (Hart, M.W., 2000). Bedding parallel shears are postulated to be a significant factor in landsliding processes. However, based on our analysis, the likelihood of these features contributing to sliding within the property limits is low provided that mitigative measures are incorporated in slope design.

5. GROUNDWATER

A permanent groundwater table was not encountered during our field investigation and is not anticipated to significantly impact project development as presently proposed. It is not uncommon for groundwater or seepage conditions to develop where none previously existed. Surface water that is not properly drained will typically perch on the top of the impervious clay soil. Therefore, proper surface drainage of irrigation and rain runoff will be critical to future performance of the project. Seeps were observed in some of the borings and running water was encountered in the Johnson Canyon drainage bottom. The seeps encountered in the borings appear to be related to localized perched ground water conditions.

6. GEOLOGIC HAZARDS

6.1 Seismic Hazard Analysis

According to the computer program *EZ-FRISK (Version 7.65)* there are 6 known active faults located within a search radius of 50 miles from the property. We used the 2008 USGS fault database, which provides several models and combinations of fault data to evaluate fault information. The nearest active faults are the Newport-Inglewood and Rose Canyon Fault Zones, located approximately 12 miles west of the site and are the dominant source of seismic ground motion. Earthquakes that might occur on the Newport-Inglewood and Rose Canyon Fault Zones or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood and Rose Canyon Fault Zones are 7.5 and 0.25g, respectively. Table 6.1.1 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 USGS 2008*, *Campbell-Bozorgnia (2008) NGA USGS 2008*, and *Chiou-Youngs (2007) NGA USGS 2008* acceleration-attenuation relationships.

TABLE 6.1.1
DETERMINISTIC SEISMIC SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson, (2008) NGA USGS 2008 (g)	Campbell-Bozorgnia, (2008) NGA USGS 2008 (g)	Chiou-Youngs, (2007) NGA USGS 2008 (g)
Newport-Inglewood/Rose Canyon	12	7.5	0.25	0.20	0.25
Rose Canyon	12	6.9	0.21	0.17	0.19
Coronado Bank	19	7.4	0.20	0.14	0.17
Palos Verdes/Coronado Banks	19	7.7	0.21	0.15	0.19
Elsinore	41	7.85	0.14	0.09	0.11
Earthquake Valley	44	6.8	0.08	0.06	0.05

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 mapped Quaternary fault is proportional to the slip rate. The program accounts for earthquake magnitude as a function of fault rupture length, 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 USGS 2008* in the analysis. Table 6.1.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 6.1.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, (2008) NGA USGS 2008 (g)	Campbell-Bozorgnia, (2008) NGA USGS 2008 (g)	Chiou-Youngs, (2007), NGA USGS 2008 (g)
2% in a 50 Year Period	0.41	0.34	0.39
5% in a 50 Year Period	.031	0.25	0.28
10% in a 50 Year Period	0.23	0.20	0.21

The California Geologic Survey (CGS) provides a program that calculates the ground motion for a 10 percent of probability of exceedence in a 50-year period based on an average of several attenuation relationships. Table 6.1.3 presents the calculated results from the Probabilistic Seismic Hazards Mapping Ground Motion Page from the CGS website.

TABLE 6.1.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Formational soil	Calculated Acceleration (g) Fill
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 frequency and duration of motion and soil conditions underlying the site. Seismic design of the structures should be evaluated in accordance with the most current adopted guidelines of the California Building Code (CBC).

6.2 Liquefaction

Due to the lack of a permanent near-surface groundwater table and the dense nature of proposed compacted fill and the soil of the Old Terrace Deposits and Otay Formation, the risk associated with liquefaction hazard at the site is low.

6.3 Tsunamis and Seiches

The site is located approximately 12 miles from the Pacific Ocean at an elevation of more than 520 feet above Mean Sea Level. The risk associated with inundation hazard due to tsunamis is low.

The site is not located downstream from any large bodies of water. Therefore, risk associated with inundation hazard due to seiche is low.

6.4 Landslides

Based on our review of the referenced geologic literature and our previous investigations on the property, landslide deposits have not been mapped on the site. The risk associated with ground movement hazard due to landslide is low.

6.5 Subsidence and Seismic Settlement

Based on the subsurface conditions encountered during our field investigation, the risk associated with ground subsidence or seismic settlement hazard is low.

6.6 Flooding

The site is not located within an active drainage or floodplain; therefore, the risk associated with inundation hazard due to flooding is low.

6.7 Expansive Soil

Based on our experience and laboratory testing performed at the site and in nearby projects, existing topsoil, alluvium/colluvium and the clayey soil of the Otay Formation, exhibit a *high* to *very high* expansion potential (Expansion Index higher than 90). The Old Terrace Deposits and the sandy soil of the Otay Formation exhibits *low to medium* expansion potential (Expansion Index from 21 to 90).

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 No soil or geologic conditions were encountered that would preclude the proposed development, provided the recommendations presented herein are implemented in design and construction of the project.

7.1.2 Our field investigation indicates that the site is underlain by weak and highly expansive claystones and potentially compressible, undocumented fill soils, topsoils, alluvial/colluvial deposits that will require special consideration during grading operations. Formational soils of the Old Terrace Deposits and Otay Formation underlie the surficial materials and extend to the maximum depth of exploration. The undocumented fill soils, topsoils, alluvial/colluvial deposits and the weathered soil of the Otay Formation are unsuitable in their present condition to receive settlement-sensitive improvements and/or additional structural fill soils. The remedial grading recommendations presented in the *Grading* section should be closely followed to properly compact the surficial soils. The soils of the Old Terrace Deposits and unweathered Otay Formation should provide adequate soil support characteristics in their natural state and where placed as properly-compacted fill.

7.1.3 Weak, highly-expansive, bentonitic claystones may be present within 10 feet of subgrade in Lot 17. Bentonite claystones exposed within 10 feet of proposed grade on the sheet graded lots and 6 feet from subgrade in proposed road ways should be removed and replaced with *low*-expansive materials.

7.1.4 We expect anticipated that weak claystones may be present on some of the cut slopes that may require stabilization measures in the form of buttresses or stability fills. Cut slopes should be observed by our project Engineering Geologist during grading operations to check that the soil and geologic conditions are as anticipated in this report.

7.1.5 The undocumented fill soils contain considerable amounts of trash and debris. Extensive sorting and/or export of these soils should be anticipated during grading operations.

7.1.6 The cut operations in the area underlain by Old Terrace Deposits will generate oversize rocks that will require special handling and placement. All oversize materials should be placed in accordance with the grading specifications contained in Appendix C.

7.1.7 Highly expansive soils will be encountered within the topsoils, alluvial and alluvial/colluvial deposits as well as in the soils of the Otay Formation. Highly expansive soils should be placed in the deeper portions of the fill areas. We expect, however, that there are sufficient *low* to *medium* expansive soils available for capping purposes on the site to mitigate the adverse impact of expansive soils.

7.1.8 Perched groundwater may be present within the low-lying alluvial/colluvial areas. Hence, remedial measures in the form of subdrains may be required where filling of the drainage courses is planned. The need for subdrains will be determined upon our review of the final grading plan.

7.1.9 In general, the undisturbed soils are expected to exhibit low erosion potential. However, fill areas or areas stripped of native vegetation will require special consideration to reduce the erosion potential. In this regard, desilting basins, improved surface drainage, and early planting of erosion-resistant ground covers are recommended.

7.1.10 Subsurface conditions observed may be extrapolated to reflect general soil and geologic conditions; however, variations in subsurface conditions between trench and boring locations should be anticipated. The Geologic Map, attached as Figure 2, presents the areal extent of the geologic conditions encountered. Cross-sections A-A', B-B', C-C', D-D', Figures 3 and 4, respectively, present the general soil conditions encountered.

7.1.11 No significant geologic hazard that would adversely affect he proposed project were observed or are known to exist on the site.

7.2 Soil and Excavation Characteristics

7.2.1 Onsite soils can be excavated with moderate to heavy effort with conventional heavy-duty equipment.

7.2.2 Based on our experience in the area and laboratory tests, the soil encountered during the field investigation is considered to be *expansive* (expansion index [EI] higher than 20) as defined by 2013 California Building Code (CBC) Section 1805.5.3. Table 7.2 presents soil classifications based on the Expansion Index.

TABLE 7.2
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2010 CBC Expansion Classification
0-20	Very Low	Non-Expansive
21-50	Low	Expansive
51-90	Medium	
91-130	High	
Greater than 130	Very High	

7.3 Temporary Excavations

7.3.1 Temporary excavations should be conducted in conformance with OSHA requirements. Existing undocumented fill, topsoil, alluvium/colluvium and the weathered soil of the Otay Formation can be considered Type B soil in accordance with OSHA guidelines. The Old Terrrace Deposits and the Otay Formation can be considered Type A soil. In general, special shoring will not be necessary if temporary excavations are less than 3 feet high. Temporary excavation depths greater than 3 feet should be laid back at an appropriate inclination or shored. The soils exposed in these excavations should not become saturated or allowed to dry. 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 or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

7.4 Slope Stability

7.4.1 Slope stability analyses using laboratory shear strength information and experience with similar soil conditions in nearby areas indicate that 2:1 (horizontal:vertical) fill slopes constructed of on-site granular materials should have calculated factors of safety of at least 1.5 under static conditions for both deep-seated failure and shallow sloughing conditions for heights of 40 feet. The 2:1 cut slopes are expected to be excavated predominantly in the Otay Formation. Based on the calculations and experience with similar conditions, 2:1 cut slopes to the planned heights should possess a factor of safety of at least 1.5 with respect to slope stability if free of adversely oriented bedding, joints or fractures. Slope stability calculations for deep-seated and surficial stability conditions are presented on Figures 5 through 8.

7.4.2 Keying and benching operations during grading of the slopes should be performed in accordance with Appendix C. Due to the presence of highly weathered Otay Formation at some locations, keying operations may extend deeper than normal (on the order of 3 to 5 feet).

7.4.3 Cut slopes within the Otay Formation may require further evaluation due to the possible presence of claystone and siltstone lenses. Stability fills may be necessary to prevent surficial sloughage of the slope faces. The potential presence of bentonitic clay lenses and the associated slope stability considerations can be addressed at the time of grading.

7.4.4 We recommend that all cut slope excavations be observed during grading by our engineering geologist to check that soil and geologic conditions do not differ significantly from those anticipated.

7.4.5 The outer 15 feet (or a distance equal to the height of the slope, whichever is less) of fill slopes should be composed of properly compacted granular "soil" fill to reduce the potential for surficial sloughing. In general, soils with an Expansion Index of less than 90 or at least 35 percent sand size particles should be acceptable as "granular" fill. Slopes should be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet and should be track-walked at the completion of each slope such that the fill soils are uniformly compacted to at least 90 percent relative compaction to the face of the finished slope.

7.4.6 All slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. In addition, all slopes should be drained and properly maintained to reduce erosion. Slope planting should generally consist of drought-tolerant plants having a variable root depth. Slope watering should be kept to a minimum to just support the plant growth.

7.5 Bulking and Shrinkage

7.5.1 Estimates of embankment bulking and shrinkage factors are typically based on comparing laboratory compaction tests with the density of the material in its natural state as encountered in the test borings and trenches. Variations in existing soil density, as well as in compacted fill densities, render shrinkage value estimates very approximate. As an example, the contractor can compact the fill soils to any relative compaction of 90 percent or higher of the maximum laboratory density. Thus, the contractor has approximately a 10 percent range of control over the fill volume. Based on our experience on nearby sites, in our opinion the following shrinkage factors can be used as a basis for estimating how much the on-site soils may shrink or swell (bulk) when excavated from their existing state and placed as compacted fills.

TABLE 7.5
SHRINKAGE AND BULK FACTORS

Soil Unit	Shrink/Bulk Factor
Undocumented Fill Soil	15 to 20 percent Shrink
Topsoil, Alluvium/Colluvium	10 to 15 percent Shrink
Otay Formation	5 to 10 percent Bulk
Old Terrace Deposits	10 to 15 percent Bulk

7.6 Grading

7.6.1. All grading should be performed in accordance with the *Recommend Grading Specifications* contained in Appendix C and the County of San Diego Grading Ordinances. Where the recommendations of Appendix C conflict with this section of the report, the recommendations of this section take precedence.

7.6.2 Earthwork should be observed by, and compacted fill tested by, representatives of Geocon Incorporated.

7.6.3 A preconstruction conference should be held at the site prior to the beginning of grading operations with the developer, contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

7.6.4 Site preparation should begin with the removal of all deleterious matter and vegetation. The depth of removal should be such that material to be used in fills is free of organic matter. Any existing underground improvements (not projected to remain should be removed and the resulting depressions properly backfilled in accordance with the procedures described herein. Material generated during stripping operations and/or site demolition should be exported from the site.

7.6.5 All undocumented fill, topsoils, and colluvial/alluvial deposits not removed by planned grading should be removed to firm natural ground and properly compacted to at least 90 percent of the maximum dry density as determined by ASTM D 1557 at moisture contents 1 to 3 percent above optimum.

7.6.6 The upper 5 to 8 feet of the Otay Formation is highly weathered and will require removal and compaction as compacted fill. The actual depth of removal will be evaluated in the field during grading operations.

7.6.7 After all unsuitable soils and deleterious material have been removed, areas planned to receive structural fill soils and/or settlement-sensitive improvements should be scarified to a depth of approximately 12 inches, moisture conditioned to 1 to 3 percent above optimum moisture content, and recompacted to a minimum of 90 percent of the dry density determined by ASTM D 1557.

7.6.8 The site should then be brought to final subgrade elevations with structural fill compacted in layers. In general, native site soils are suitable for reuse as fill if free from vegetation, debris and other deleterious matter. Layers of fill should be no thicker than will allow for adequate bonding and compaction. All fill (including backfill and scarified ground surfaces) should be compacted to at least 90 percent of maximum dry density at optimum moisture content or above, as determined in accordance with the ASTM D 1557, at moisture contents ranging from 1 to 3 percent above the optimum content. Fill soils placed at moisture contents outside this range of moisture content may be considered unacceptable at the discretion of the geotechnical engineer.

7.6.9 Highly-expansive soils (EI >90) should not be placed within the upper 5 feet of finished pad grade. Bentonite with *critically high* expansive potential should not be placed within 10 feet of finish grade. Similarly, cut lots containing highly expansive soils within 5 feet of finish grade should be undercut 5 feet and capped with *low* to *medium* (EI between 21 and 90) expansive materials.

7.6.10 Where bentonite materials are present within 10 feet of finish grade on cut lots, this condition should be evaluated on an individual lot basis and mitigative measures provided in updated geotechnical reports once building location and anticipated structural loading are determined.

7.7 Seismic Design Criteria

7.7.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 7.7.1 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 building structure and improvements should be designed using a Site Class D. 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 7.7.1 are for the risk-targeted maximum considered earthquake (MCE_R). The values presented in Table 7.7.1 are for preliminary purposes. Once specific grading plans with building locations are developed for each lot, Geocon Incorporated should be contracted to provide specific seismic design criteria.

TABLE 7.7.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.808 g	Figure 1613..3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.310 g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.177	Table 1613.3.3(1)
Site Coefficient, F_V	1.780	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.951 g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.552 g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.634 g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.368 g	Section 1613.3.4 (Eqn 16-40)

7.7.2 Table 7.7.2 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 7.7.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.313 g	Figure 22-7
Site Coefficient, F_{PGA}	1.187	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.372 g	Section 11.8.3 (Eqn 11.8-1)

7.7.3 Conformance to the criteria in Tables 7.7.1 and 7.7.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid all damage, since such design may be economically prohibitive.

7.8 Foundation Recommendations

7.8.1 Continuous footings or isolated spread footings for one- and/or two-story structures should be at least 12 inches wide and should extend at least 18 inches below lowest adjacent pad grade into properly compacted fill soils as recommended in Section 7.6. Isolated spread footings for one- and/or two-story structures should be at least 2 feet wide and extend 18 inches below lowest adjacent pad grade into properly compacted fill soils. Figure 9 presents a footing dimension detail depicting lowest adjacent grade. Minimum continuous footing reinforcement for one- and/or two-story structures should consist of four No. 4 steel-reinforcing bars placed horizontally in the footings; two near the top and two near the bottom.

7.8.2 The recommended dimensions and steel reinforcement presented above are based on soil characteristics only and are not intended to be in lieu of reinforcement necessary to satisfy structural loading. Actual reinforcement of the foundations should be designed by the project structural engineer.

7.8.3 The recommended allowable bearing capacity for foundations designed as recommended above is 2,500 pounds per square foot for 18-inch-deep footings. This value is for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

7.8.4 Footing excavations should be observed by a representative of Geocon Incorporated prior to placing reinforcing steel to verify that soil conditions are similar to those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.9 Concrete Slabs-on-Grade

7.9.1 Interior concrete slabs-on-grade should be at least 5 inches thick. Where heavy concentrated floor loads are anticipated, the slab thickness should be increased to 6 inches and should be underlain by 4 inches of Class 2 base material compacted to at least 95 percent relative compaction. The allowable soil bearing pressure under slabs with import, *low* expansive soils is 1,500 pounds per square foot.

7.9.2 Minimum reinforcement of slabs-on-grade placed on *low* to *medium* expansive soil should consist of No. 3 reinforcing bars placed at 18 inches on center in both horizontal directions. The concrete slabs-on-grade should also be doweled into the foundation system to prevent vertical movement between the slabs, footings, and walls.

7.9.3 The concrete slab-on-grade recommendations are minimums based on soil support characteristics only. We recommend that the project structural engineer evaluate the structural requirements of the concrete slabs for supporting equipment and storage loads.

7.9.4 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). In addition, the membrane should be installed in accordance with manufacturer's recommendations and ASTM requirements and installed in a manner that prevents puncture. The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity controlled environment.

7.9.5 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. It is common to have 3 to 4 inches of sand for 5-inch thick slabs in the southern California region. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

7.9.6 All exterior concrete flatwork not subject to vehicular traffic should be a minimum of 4 inches thick and conform to the following recommendations. Slab panels in excess of 8 feet square should be reinforced with 6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh to reduce the potential for cracking. In addition, all concrete flatwork should be provided with crack-control joints to reduce and/or control shrinkage cracking. Crack-control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack-control spacing. Subgrade soils for exterior slabs should be compacted in accordance with criteria presented in the grading section of this report. The subgrade soils should not be allowed to dry prior to placing concrete.

7.9.7 The recommendations presented herein are intended to reduce the potential for cracking of slabs and foundations as a result of differential soil movement. However, even with the incorporation of these recommendations, foundations and slabs-on-grade will still exhibit some cracking. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack-control joints and proper concrete placement and curing. Crack-control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Cement Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

7.10 Lateral Loads for Retaining Walls

7.10.1 Retaining walls 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 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 pcf. Where the backfill will be inclined at 2:1 (horizontal:vertical), an active soil pressure of 50 pcf is recommended. Expansive soil should not be used as backfill material behind retaining walls. Soil placed for retaining wall backfill should have an Expansion Index less than 50. Existing soils exhibited a *low* to *high* expansion potential. Therefore, stockpiling of *low* expansive soils encountered during grading or import of *low* expansive granular soil may be required for retaining wall backfill.

7.10.2 Where walls are restrained from movement at the top, an active soil pressure equivalent to the pressure exerted by a fluid density of 60 pcf should be used for horizontal backfill. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added (unit weight 125 pcf).

7.10.3 Soil contemplated for use as retaining wall backfill should be identified in the field prior to backfilling. At that time, Geocon Incorporated should obtain samples for laboratory testing to evaluate its suitability. Modified lateral earth pressures may be necessary if the backfill soil does not meet the required expansion index or shear strength. County of San Diego or regional standard wall designs, if used, are based on a specific active lateral earth pressure and/or soil friction angle. In this regard, onsite soil to be used as backfill may or may not meet the values for standard wall designs. Geocon Incorporated should be consulted to assess the suitability of the onsite soil for use as wall backfill if standard wall designs will be used.

7.10.4 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. 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 structures adjacent to the base of the wall. The above recommendations assume a properly compacted granular (EI of less than 50) free-draining backfill material with no hydrostatic forces or imposed surcharge load. A typical retaining wall drainage detail is presented on Figure 10, attached. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

7.10.5 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 1803.5.12 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 17H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.372g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

7.10.6 To resist lateral loads, a passive pressure equivalent to the pressure exerted by a fluid density of 300 pcf should be used for design of footings or shear keys poured neat against properly compacted granular fill soils. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

7.10.7 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.4 should be used for design. To resist lateral loads, the passive resistance can be combined with friction.

7.10.8 The recommendations presented above are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 8 feet. In the event that walls higher than 8 feet are planned, Geocon Incorporated should be consulted for additional recommendations.

7.11 Preliminary Pavement Recommendations

7.11.1 The following recommendations are for preliminary purposes and are provided for private driveways and parking areas. The final pavement section design will depend upon soil conditions exposed at subgrade elevation and the results of Resistance Value (R-Value) tests. The following preliminary pavement section recommendations are based on an assumed R-Value of 15. Sections are presented for both flexible (asphalt concrete) and rigid (Portland cement concrete) pavement.

7.11.2 The pavement sections for public streets will be determined by the County of San Diego Materials Testing and Engineering Department. The final pavement sections of public streets will be dependent on the traffic index designated by the County of San Diego Materials Testing and Engineering Department and the R-Value laboratory test results of the exposed subgrade soils.

TABLE 7.11.1
FLEXIBLE PAVEMENT SECTIONS RECOMMENDATIONS

Location	Assumed Traffic Index (TI)	Assumed R-Value	Asphalt Concrete Thickness (inches)	Class 2 Aggregate Base Thickness (inches)
Parking stalls for automobiles and light-duty vehicles	4.5	15	3	6.0
Driveways for automobiles and light-duty vehicles	5.5	15	3	10.0
Driveways and parking areas for heavy-duty trucks and fire lanes	7.0	15	4	13.0

TABLE 7.11.2
RIGID PAVEMENT SECTIONS RECOMMENDATIONS

Location	Average Daily[1] Truck Traffic (ADTT assumed)	Assumed R-Value	Portland Cement Concrete[2] (inches)	Class 2 Aggregate Base Thickness (inches)
Parking stalls[3] for automobiles and light-duty vehicles	25-100	20	5	4
Driveways[3] for automobiles and light-duty vehicles	300-500	20	6[†]	4
Driveways and parking areas for heavy-duty trucks and fire lanes	100-500	20	7[‡]	4

[1]ADTT values have been assumed for planning purposes herein and should be confirmed by the design team during future plan development.

[2]Concrete shall have a minimum $M_R \geq 600$ psi**.** This analysis assumes the construction of concrete shoulders.

[3]Parking stalls and driveways assume typical light truck and car traffic.

[†]Slabs should be reinforced with No. 3 reinforcing bars at 24 inches on center in both horizontal directions.

[‡]Slabs should be reinforced with No. 4 reinforcing bars at 24 inches on center in both horizontal directions.

7.11.3 The subgrade soils should be compacted to a minimum relative compaction of 95 percent at 1 to 3 percent above the optimum moisture content. The depth of subgrade compaction should be approximately 12 inches.

7.11.4 Class 2 base should conform to Section 26-1.-02B of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* and should be compacted to a minimum of 95 percent of the maximum dry density at near optimum moisture content. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Green Book)*.

7.11.5 Where trash bin enclosures are planned within asphalt paved areas, we recommend that the pavement sections be equivalent to the heavy-duty truck categories presented in the respective tables. The concrete should extend into the roadway sufficiently so that all wheels of the trash truck are on the concrete when loading.

7.11.6 Rigid Portland cement concrete sections were evaluated using methods suggested by the American Concrete Institute *Guide for Design and Construction of Concrete Parking Lots (ACI330R-08).*

7.11.7 Construction joints should be provided at a maximum spacing of 12 feet each way to control shrinkage. Installation of these types of joints should be made immediately after concrete finishing.

7.11.8 Construction jointing, doweling, and reinforcing should be provided in accordance with recommendations of the American Concrete Institute.

7.11.9 The performance of asphalt concrete pavements and Portland cement concrete pavements is highly dependent upon providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement will likely result in pavement distress and subgrade failure. If planter islands are proposed, the perimeter curb should extend at least 12 inches below proposed subgrade elevations. In addition, the surface drainage within the planter should be such that ponding will not occur.

7.11.10 Our experience indicates that even with these provisions, a groundwater condition can develop as a result of increased irrigation, landscaping and surface runoff.

7.12 Bio-Retention Basin and Bio-Swale Recommendations

7.12.1 The site will be underlain by compacted fill, Old Terrace Deposits and Otay Formation. Based on our experience with the onsite soils and infiltration testing in nearby projects, the onsite soil has very low permeability and generally very low infiltration characteristics. It is our opinion the existing soil is unsuitable for infiltration of storm water runoff.

7.12.2 Any bio-retention basins, bioswales, and bio-remediation areas should be designed by the project civil engineer and reviewed by Geocon Incorporated. Typically, bioswales consist of a surface layer of vegetation underlain by clean sand. A subdrain should be provided beneath the sand layer. Water should not be allowed to infiltrate adjacent to the planned improvements. We recommend that retention basins, be properly lined to prevent water infiltration into the underlying soil. Prior to discharging into the storm drain pipe or other approved outlet structure, a seepage cutoff wall should be constructed at the interface between the subdrain and storm drainpipe. The concrete cut-off wall should extend at least 6 inches beyond the perimeter of the gravel-packed subdrain system. Figure 11 presents a typical bioswale detail.

7.12.3 The landscape architect should be consulted to provide the appropriate plant recommendations if a vegetated swale is to be implemented. If drought resistant plants are not used, irrigation may be required.

7.13 Drainage and Maintenance

7.13.1 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 adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2013 CBC 1803.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into storm drains and conduits that carry runoff away from the proposed structure.

7.13.2 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.

7.13.3 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.

7.14 Grading and Foundation Plan Review

7.14.1 Geocon Incorporated should review the grading plans and foundation plans prior to final design submittal to determine if additional analysis and/or recommendations are required.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. 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.

2. 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.

3. This report is issued with the understanding that it is the responsibility of the owner or 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.

4. The findings of this report are valid as of the present date. 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.

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VICINITY MAP

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EAST OTAY MESA CENTER MIXED-USE
SAN DIEGO, CALIFORNIA

RG / RS		DSK/GTYPD	DATE 07 - 20 - 2015	PROJECT NO. 06263 - 42 - 03	FIG. 1

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Qal/Qc ....ALLUVIUM/COLLUVIUM (Undifferentiated)

Qt ....OLD TERRACE DEPOSITS (Dotted Where Buried)

To ....OTAY FORMATION

B-11 ....APPROX. LOCATION OF LARGE DIAMETER BORING (FEB 26, 1999)

T-26 ....APPROX. LOCATION OF TRENCH (FEB 26, 1999)

D D' ....APPROX. LOCATION OF GEOLOGIC CROSS-SECTION

........APPROX. LOCATION OF GEOLOGIC CONTACT (Dotted Where Buried; Queried Where Uncertain)

GEOLOGIC MAP

EAST OTAY MESA CENTER MIXED-USE
SAN DIEGO, CALIFORNIA

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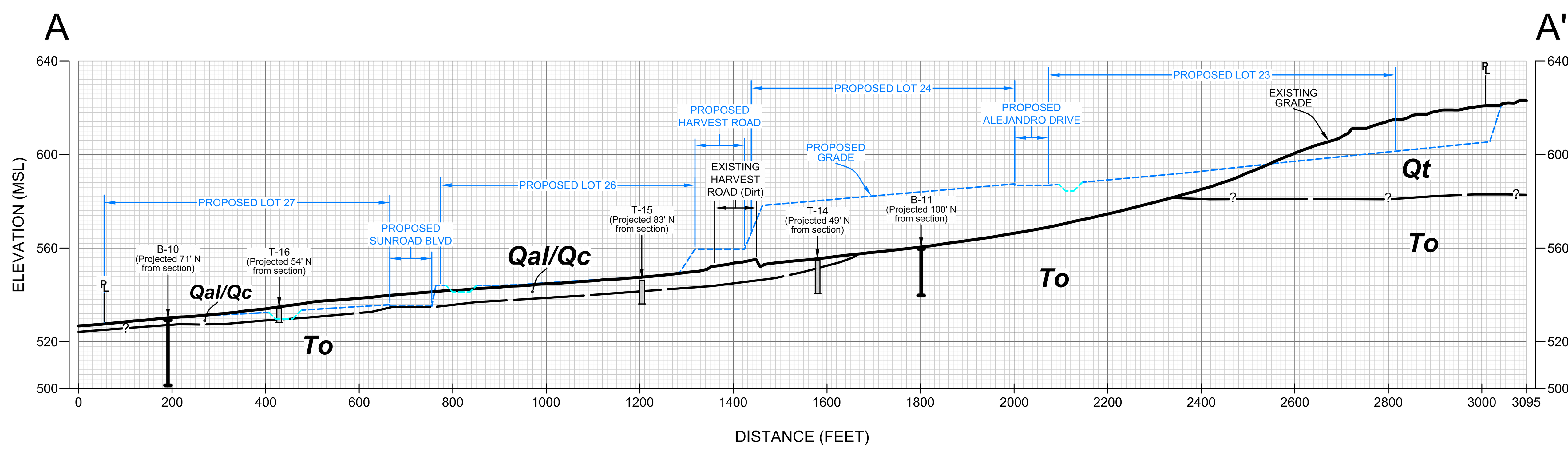
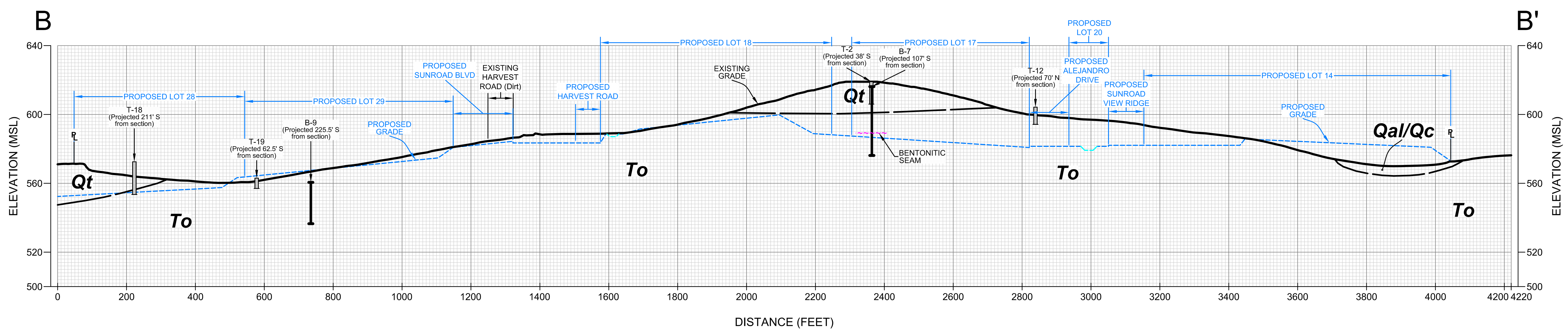
SCALE 1" = 200'	DATE 07 - 20 - 2015	
PROJECT NO. 06263 - 42 - 03		FIGURE 2
SHEET 1 OF 1		

Plotted:07/20/2015 1:21PM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\06263-42-03 East Otay Mesa Center Mixed Use\SHEETS\06263-42-03 GeoMap.dwg



GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 200' (Horiz.); 1" = 40' (Vert.)



GEOLOGIC CROSS-SECTION B-B'

SCALE: 1" = 200' (Horiz.); 1" = 40' (Vert.)

GEOCON LEGEND

- ***Qal/Qc*** ....ALLUVIUM/COLLUVIUM (Undifferentiated)
- ***Qt*** ....OLD TERRACE DEPOSITS
- ***To*** ....OTAY FORMATION
- B-11 ....APPROX. LOCATION OF LARGE DIAMETER BORING (FEB 26, 1999)
- T-20 ....APPROX. LOCATION OF TRENCH (FEB 26, 1999)
- ....APPROX. LOCATION OF PROPOSED GRADE
- ....APPROX. LOCATION OF PROPOSED DESILTIN BASIN
- ? ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)

GEOLOGIC CROSS - SECTIONS

EAST OTAY MESA CENTER MIXED-USE
SAN DIEGO, CALIFORNIA

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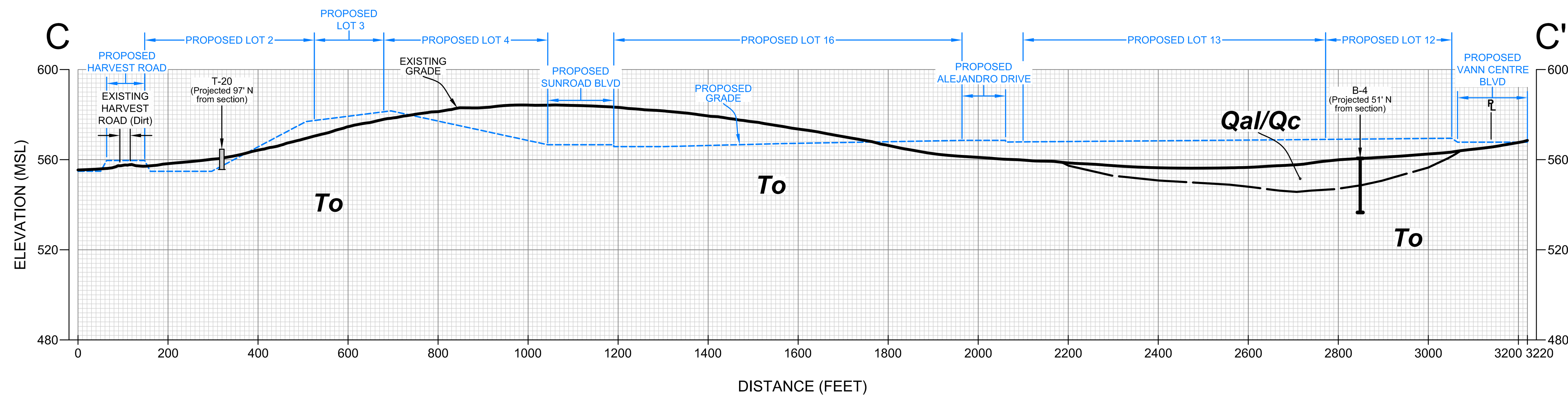
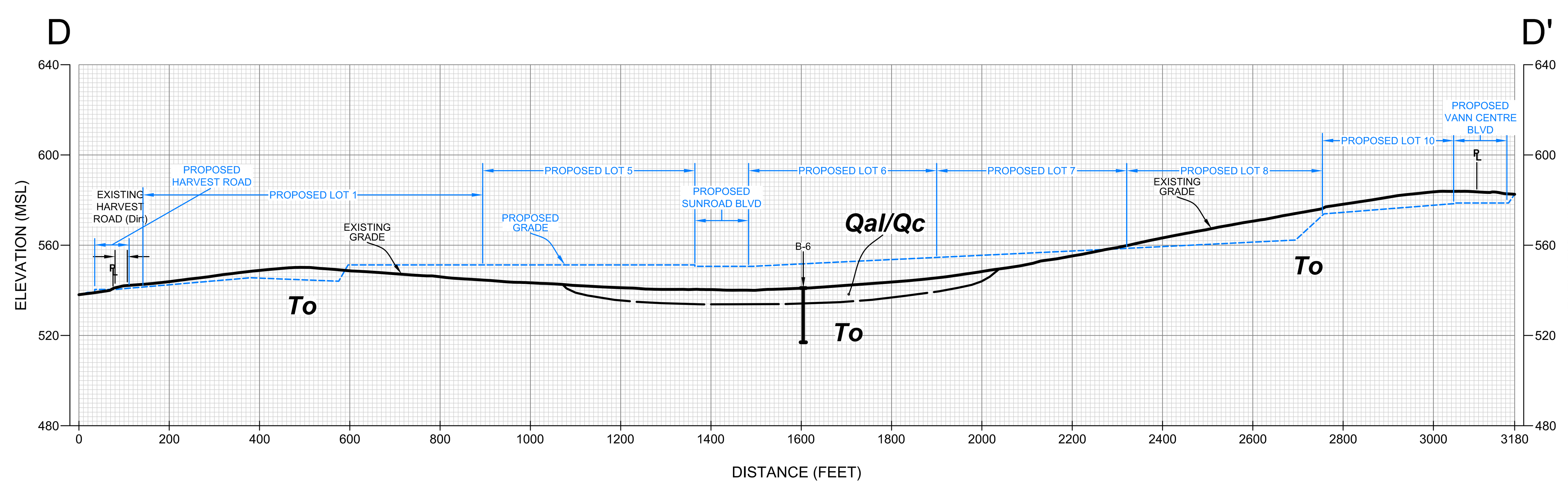
SCALE 1" = 200' (Horiz.) 1" = 40' (Vert.)	DATE 07 - 20 - 2015	
PROJECT NO. 06263 - 42 - 03		FIGURE 3
SHEET 1 OF 1		

Plotted:07/20/2015 1:25PM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\06263-42-03 East Otay Mesa Center Mixed Use\SHEETS\06263-42-03 Cross-Sections.dwg



GEOLOGIC CROSS-SECTION C-C'

SCALE: 1" = 200' (Horiz.); 1" = 40' (Vert.)



GEOLOGIC CROSS-SECTION D-D'

SCALE: 1" = 200' (Horiz.); 1" = 40' (Vert.)

GEOCON LEGEND

- ***Qal/Qc*** ....ALLUVIUM/COLLUVIUM (Undifferentiated)
- ***Qt*** ....OLD TERRACE DEPOSITS
- ***To*** ....OTAY FORMATION
- B-11 ....APPROX. LOCATION OF LARGE DIAMETER BORING (FEB 26, 1999)
- T-20 ....APPROX. LOCATION OF TRENCH (FEB 26, 1999)
- ....APPROX. LOCATION OF PROPOSED GRADE
- ....APPROX. LOCATION OF PROPOSED DESILTIN BASIN
- ? ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)

GEOLOGIC CROSS - SECTIONS

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SCALE 1" = 200' (Horiz.) 1" = 40' (Vert.)	DATE 07 - 20 - 2015
PROJECT NO. 06263 - 42 - 03	FIGURE 4
SHEET 1 OF 1	

Plotted:07/20/2015 1:26PM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\06263-42-03 East Otay Mesa Center Mixed Use\SHEETS\06263-42-03 Cross-Sections.dwg

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 40 feet
SLOPE INCLINATION	2 : 1	(Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 118.3 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 35 degrees
APPARENT COHESION	C	= 150 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$\gamma_{c\phi}$	=	$\frac{\gamma_t H \tan\phi}{C}$	EQUATION (3-3), REFERENCE 1
FS	=	$\frac{NcfC}{\gamma_t H}$	EQUATION (3-2), REFERENCE 1
$\gamma_{c\phi}$	=	22.1	CALCULATED USING EQ. (3-3)
Ncf	=	60	DETERMINED USING FIGURE 10, REFERENCE 2
FS	=	1.9	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

1......Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics, Series No. 46, 1954

2......Janbu, N., Discussion of J.M. Bell, Dimensionless Parameters for Homogeneous Earth Slopes, Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

SLOPE STABILITY ANALYSIS - FILL SLOPES

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RG / RS		DSK/GTYPD	DATE 07 - 20 - 2015	PROJECT NO. 06263 - 42 - 03	FIG. 5

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= Infinite
DEPTH OF SATURATION	Z	= 3 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)	
SLOPE ANGLE	i	= 26.6 degrees
UNIT WEIGHT OF WATER	γ_w	= 62.4 pounds per cubic foot
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 118.3 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 35 degrees
APPARENT COHESION	C	= 150 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE

SEEPAGE FORCES PARALLEL TO SLOPE FACE

ANALYSIS :

$$FS = \frac{C + (\gamma_t - \gamma_w)\ Z \cos^2 i \tan \phi}{\gamma_t\ Z \sin i \cos i} = 2.1$$

REFERENCES :

1......Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage*, Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62

2......Skempton, A. W., and F.A. Delory, *Stability of Natural Slopes in London Clay*, Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81

SURFICIAL SLOPE STABILITY ANALYSIS - FILL SLOPES

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RG / RS		DSK/GTYPD	DATE 07 - 20 - 2015	PROJECT NO. 06263 - 42 - 03	FIG. 6

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 40 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)	
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 132.3 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 35 degrees
APPARENT COHESION	C	= 500 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$\gamma_{c\phi} = \dfrac{\gamma_t H \tan\phi}{C}$		EQUATION (3-3), REFERENCE 1
$FS = \dfrac{N_{cf} C}{\gamma_t H}$		EQUATION (3-2), REFERENCE 1
$\lambda_{c\phi}$ =	7.0	CALCULATED USING EQ. (3-3)
N_{cf} =	25	DETERMINED USING FIGURE 10, REFERENCE 2
FS =	2.5	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

1......Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics, Series No. 46, 1954

2......Janbu, N., Discussion of J.M. Bell, Dimensionless Parameters for Homogeneous Earth Slopes, Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

SLOPE STABILITY ANALYSIS - CUT SLOPES

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EAST OTAY MESA CENTER MIXED-USE
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RG / RS		DSK/GTYPD	DATE 07 - 20 - 2015	PROJECT NO. 06263 - 42 - 03	FIG. 7

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= Infinite
DEPTH OF SATURATION	Z	= 3 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)	
SLOPE ANGLE	i	= 26.6 degrees
UNIT WEIGHT OF WATER	γ_w	= 62.4 pounds per cubic foot
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 132.3 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 35 degrees
APPARENT COHESION	C	= 350 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE

SEEPAGE FORCES PARALLEL TO SLOPE FACE

ANALYSIS :

$$FS = \frac{C + (\gamma_t - \gamma_w)\, Z \cos^2 i \tan \phi}{\gamma_t\, Z \sin i \cos i} = 4.1$$

REFERENCES :

1......Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage*, Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62

2......Skempton, A. W., and F.A. Delory, *Stability of Natural Slopes in London Clay*, Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81

SURFICIAL SLOPE STABILITY ANALYSIS - CUT SLOPES

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EAST OTAY MESA CENTER MIXED-USE

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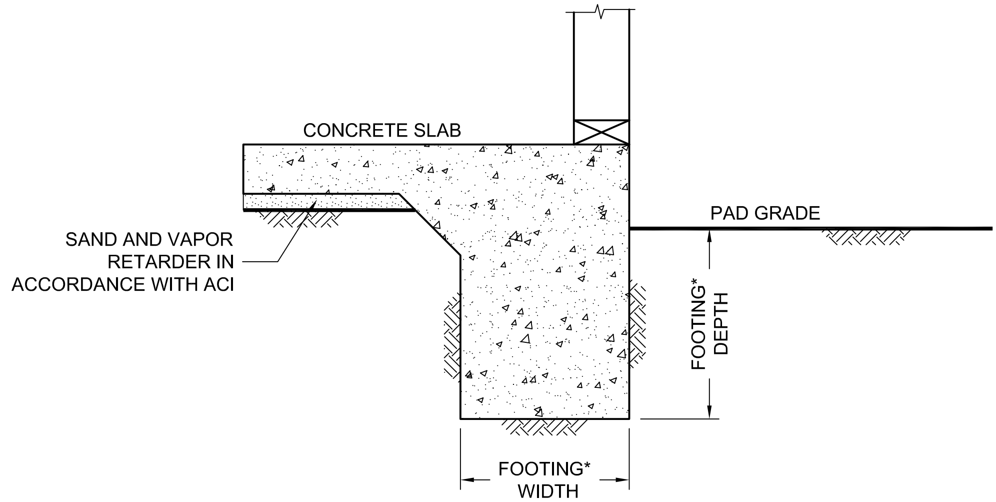
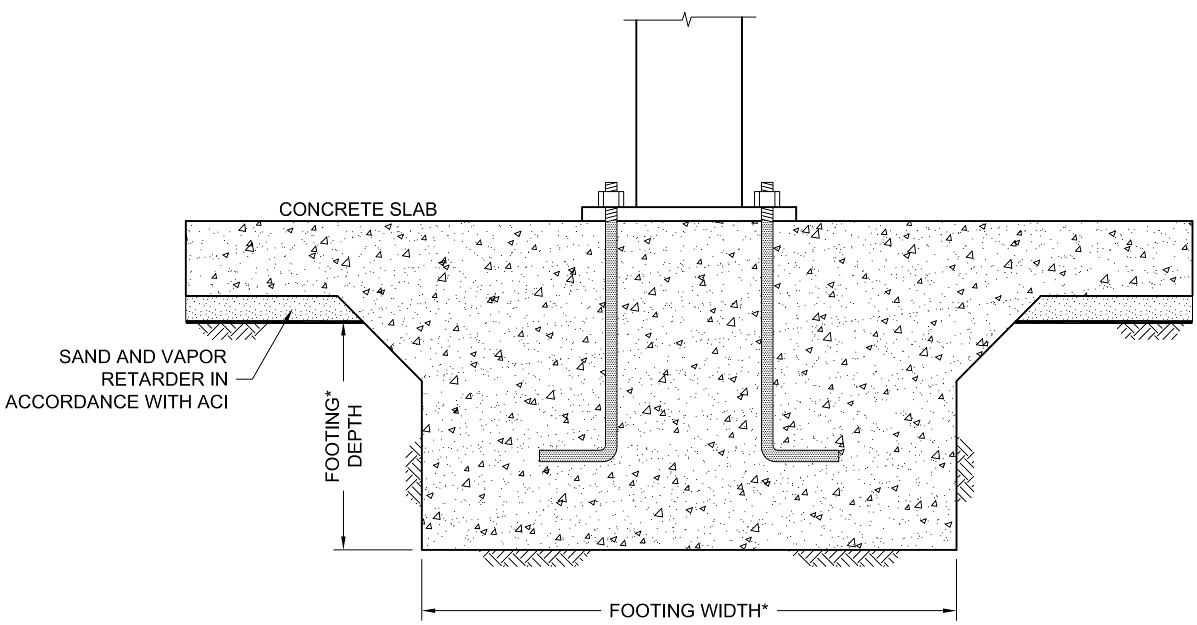
RG / RS		DSK/GTYPD	DATE 07 - 20 - 2015	PROJECT NO. 06263 - 42 - 03	FIG. 8





*....SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

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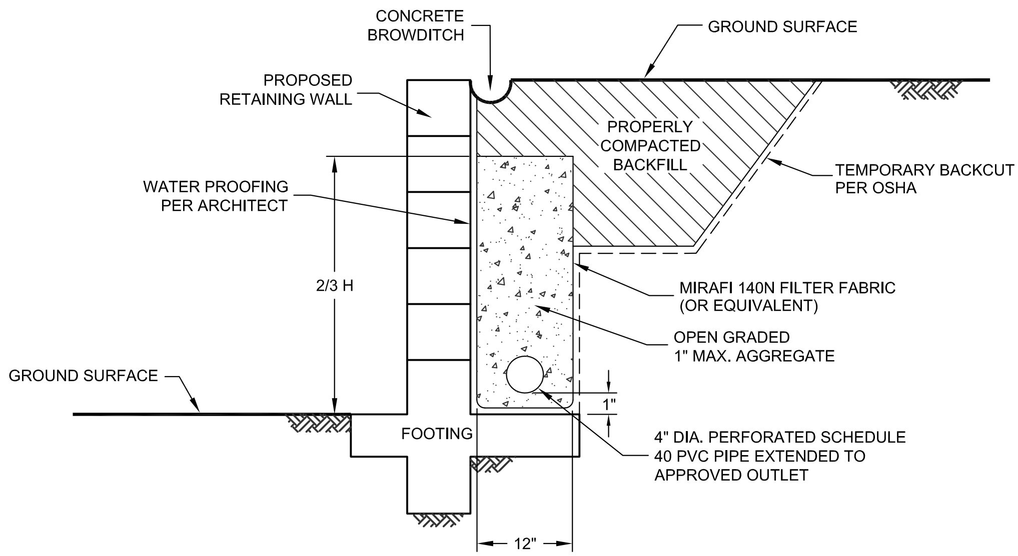
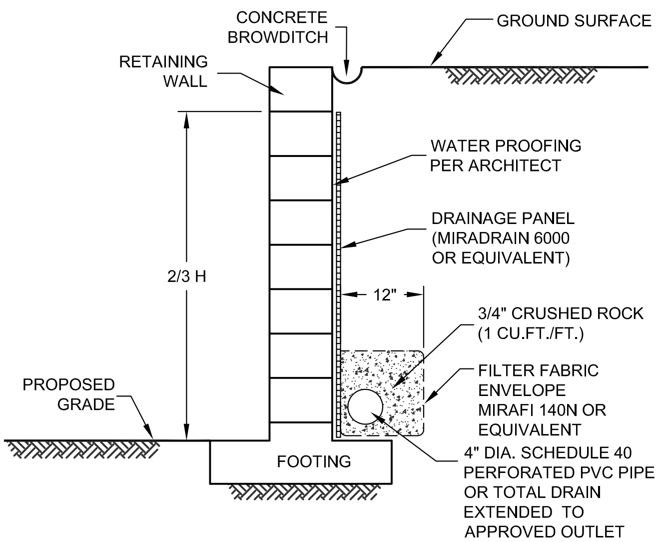
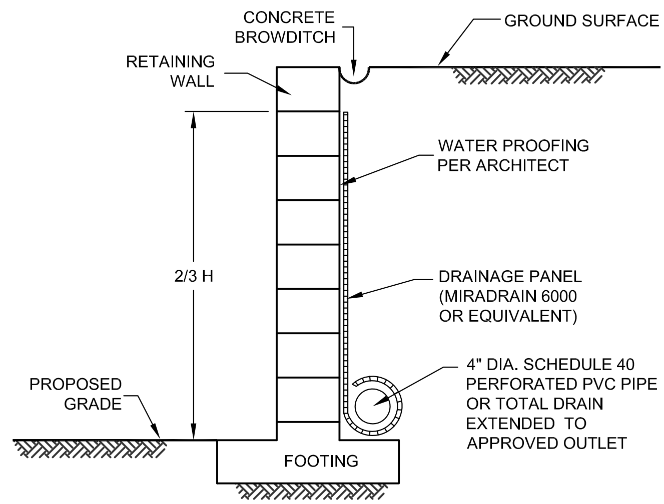
RG / RS		DSK/GTYPD	DATE 07 - 20 - 2015	PROJECT NO. 06263 - 42 - 03	FIG. 9







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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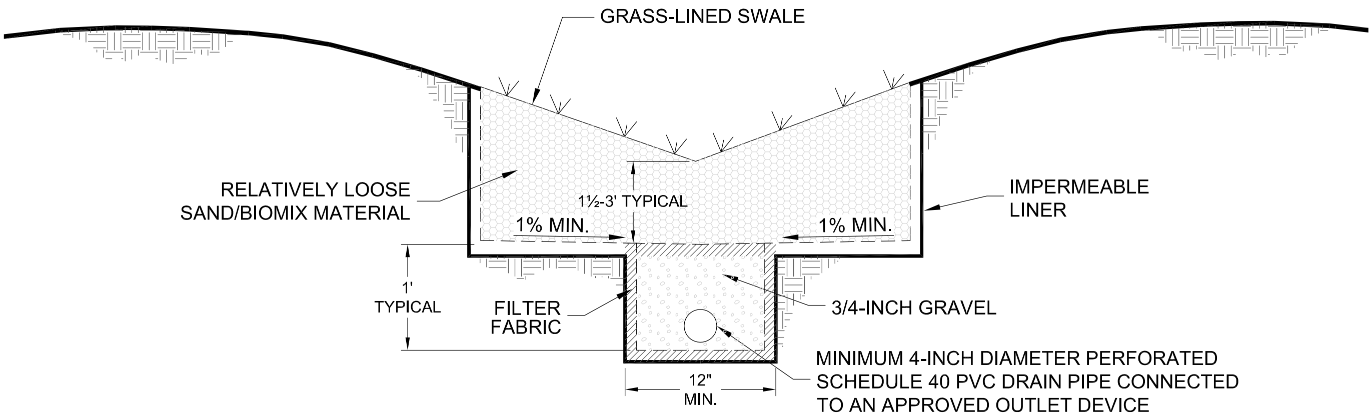
RG / RS		DSK/GTYPD	DATE 07 - 20 - 2015	PROJECT NO. 06263 - 42 - 03	FIG.10



NO SCALE

VEGETATED SWALE BIOFILTER DETAIL

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RG / CW		DSK/GTYPD	DATE 07 - 20 - 2015	PROJECT NO. 06263 - 42 - 03	FIG. 11

Y:\PROJECTS\06263-42-03 East Otay Mesa Center Mixed Use\DETAILS\VSBD_2 (Bioswale).dwg

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed between September 7 and September 20, 1990, and consisted of geologic mapping of 11 large-diameter exploratory borings and 26 exploratory trenches at the approximate locations shown on the attached Geologic Map, Figure 2 (Map Pocket). The borings were advanced to depths ranging from 20 feet to 90 feet below existing grade utilizing an E100 drill-rig equipped with a 30-inch-diameter bucket auger. The trenches were excavated utilizing a John Deere 710 backhoe and/or a John Deere 555 trackhoe.

Relatively undisturbed samples were obtained from the borings by driving a three-inch O. D. split-tube sampler into the soil mass with blows from the drill rig's Kelly bar falling 12 inches. The sampler was equipped with 1-inch by 2⅜-inch brass sampler rings to facilitate removal and testing. Disturbed samples of prevailing soils were also obtained from the borings and trenches.

The soil conditions encountered in the trenches were visually examined, classified, and logged in general conformance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D 2844). The logs of the exploratory borings and trenches are presented on Figures A-1 through A-45. The logs depict the various soil types encountered and indicate the depths at which samples were obtained.

FILE NO. 04581-03-01

BORING B 1

ELEVATION 572 DATE COMPLETED 9/10/90

EQUIPMENT E-100 BUCKET DRILL

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0–3				CL	**TOPSOIL** Soft, dry, dark gray, Sandy CLAY			
3–8	B1-1			SM	**OTAY FORMATION** Highly weathered, fractured, dry, whitish gray Silty fine SANDSTONE interbedded with Sandy SILTSTONE	3/12"	103.3	16.1
8–10				CL	Hard, humid, fractured purplish CLAYSTONE, bedding attitude near horizontal			
10–14	B1-2			SM	Very dense, humid, light gray Silty fine SANDSTONE	3/12"	105.8	16.6
14–15	B1-4			ML	Purplish sandy siltstone from 14 to 15 feet			
15–20	B1-3			SM	Very dense, humid, light gray Silty fine SANDSTONE	5/12"	108.7	16.2
20–22.5	B1-5			ML	Very stiff to hard, humid, purplish-brown Clayey SILTSTONE. Contact gradational	3/12"	84.1	35.8
22.5–23				CH	Bentonite layer approximately 6 inches thick, attitude horizontal. Shear zone bedding plane fault 1/2 inch thick - horizontal			
23–27	B1-6			ML	Hard, humid, pinkish-gray, Clayey SILTSTONE	13/12"	125.7	10.5
27–28.5				SM	Grades into massive, gray, very fine silty sandstone at 27 feet			
28.5–30	B1-7			ML	Grades into hard, purplish siltstone	10/12"	114.8	17.1

Figure A-1 Log of Test Boring B 1, page 1 of 3

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SAMPLE SYMBOLS ☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED) ⊠ ... DISTURBED OR BAG SAMPLE ▨ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

FILE NO. 04581-03-01

BORING B 1

ELEVATION 572 DATE COMPLETED 9/10/90

EQUIPMENT E-100 BUCKET DRILL

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30				SM	at 29 feet Grades into hard, purplish siltstone at 29 feet (continued)			
32–34				SM	Very dense, moist, light gray, massive, fine Silty SANDSTONE			
35				CL	Hard claystone layer. Attitude near horizontal			
36–38				SM	Very dense, moist, light gray, massive, fine Silty SANDSTONE			
39				CL	Hard claystone bed from 38.5 to 39.5 feet			
40	B1-8			SM	Very dense, moist, light gray, massive, fine Silty SANDSTONE	20/12"	129.3	6.0
43				SP	Very hard, well-cemented sandstone from 42.5 to 43.5			
44–46				SM	Very dense, moist, light gray, massive, fine Silty SANDSTONE			
46–49				SM	Very hard, moist, massive, light gray Sandy SILTSTONE			
50	B1-9			SM	Very dense, moist, gray, massive fine Silty SANDSTONE	17/12"	106.6	20.6
56				CL	Very hard, massive, humid, purplish brown Silty CLAYSTONE			
58				CH	Very hard, purplish-gray, Bentonitic CLAY conchoidal fracturing			

Figure A-2 Log of Test Boring B 1, page 2 of 3

ECKE

SAMPLE SYMBOLS ☐... SAMPLING UNSUCCESSFUL ◧... STANDARD PENETRATION TEST ■... DRIVE SAMPLE (UNDISTURBED) ⊠... DISTURBED OR BAG SAMPLE ▨... CHUNK SAMPLE ▼... WATER TABLE OR SEEPAGE