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PRELIMINARY GEOTECHNICAL EVALUATION FOR A PROPOSED MULTI-UNIT APARTMENT BUILDING TO REPLACE THE EXISTING STRUCTURES LOCATED AT 6253-6265-6275 MONTEZUMA ROAD SAN DIEGO CALIFORNIA 92115

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<u>ATTACHMENTS</u>

INTRODUCTION

General

This report presents the results of a preliminary geotechnical evaluation for a proposed apartment building to replace the existing buildings located at 6253, 6265 and 6275 Montezuma Road, in the College area of the City of San Diego, San Diego County, California (see Figure 1, "Site Vicinity Map," and Figure 2, "Site Location Map"). For the purposes of clarity and consistency within this report, the front of the building will be assumed to face north towards Montezuma Road, and all references to direction throughout the report will be based on this assumption.

The scope of our work, conducted to date, includes the following:

- **RESEARCH**
- **FIELD EVALUATION**
- **ANALYSIS & DISCUSSION**
- **CONCLUSIONS & RECOMMENDATIONS**
- **MISCELLANEOUS**

Proposed Site Development

Based on our review of project data and conversations with the project Developer and their Architect, we understand that the proposed site development will be to demolish the existing structures on the three (3) adjacent lots described above and construct a four (4) story, thirty-one (31) unit apartment building above a parking structure that is partially below grade. The exact grades and elevations are not finalized at this point, but plans addressing preliminary suitability indicate the garage will be slightly above or below Montezuma Road. The lower level will be constructed of masonry (or shotcrete) supporting a concrete podium deck, with four (4) story wood construction above the podium. Most of the property will be covered with the footprint of the building, driveway and sidewalks, leaving very little landscaped area when the project is completed. We understand that the landscaping will consist primarily of drought tolerant plantings. The need for structural steel for vertical support, and/or flagpoles or moment frames for lateral support has not been determined and most likely won't be until the exact plan layout has been resolved. Please see Figure 3, "Schematic Site Plan/Location of Test Pits" for a depiction of the proposed site development.

Based on the proposed construction, it is likely that point loads within the garage could be as much as one hundred to one hundred fifty (100 to 150) kips. The perimeter loads would be line loads of approximately three thousand to five thousand (3,000 - 5,000) pounds per linear foot.

RESEARCH

General

Our research of the property consisted of the following review:

- Geologic Map Review
- Review of Provided Plans (Limited)
- Review of Published Documents

Geologic Map Review

We conducted a preliminary, cursory map review. A geologist was retained to conduct a complete geology study so no further review or determinations or conclusions will be addressed in this section of the report.

Review of Provided Plans (Limited)

The only plans provided to us as part of our study were a site plan and topographic plan for the project. The plan showed the existing buildings, as well as the proposed structure footprints on the site. No floor plans, roof plans, or cross sections of the project were provided to us for document review as part of this study.

Review of Published Documents

Published documents relevant to the site were reviewed as part of this study; see Appendix A.

FIELD EVALUATIONS

General

Representatives of Accutech Engineering Systems, Inc. visited the subject site on several occasions in June and July of 2018 to perform field observations. The site visits included an original trip to the site to understand the project development, another visit to stake the desired locations of the proposed test pits, and a final site visit to observe the digging of the test pits, obtain field readings of the in-place soils and obtain samples of the soils for laboratory testing. The field study included the following:

- Area and Site Reconnaissance

- Subsurface Evaluation
- Geologic Reconnaissance

Additionally, reviews of the following property features were taken into account for preparation of this report:

- Existing Site Development
- Site Drainage
- Site Distress

Area and Site Reconnaissance

The site is a near-rectangular, somewhat trapezoidal-shaped parcel of land located at 6253, 6265 and 6275 Montezuma Road in the City of San Diego, San Diego County, California. The site is bordered by a mix of residential and commercial properties to the north, east and west. To the south, residential properties, similar to the existing ones at this site exist.

The site is relatively flat with a mild slope down to the north and west towards Montezuma Road.

Subsurface Evaluation

Three (3) exploratory test pits were excavated at different locations across the three (3) lot site during our subsurface evaluation (see Figure 3, "Site Plan/Location of Exploratory Test Pits"). The test pits were excavated with a backhoe, as cobbles and other large aggregate were anticipated which would make drilling with a small diameter auger extremely difficult if not impossible. We believe that the backhoe provided us with enough information to develop geotechnical parameters for the foundation other geotechnical related improvements at the project. The test pits were located in areas that were not occupied by existing surface hardscape or overheard obstructions. Subsurface soils were reviewed and logged under the supervision of a Licensed Geotechnical Engineer and Certified Engineering Geologist during the excavations, while disturbed samples were obtained for laboratory testing. Due to the large aggregate portion, no undisturbed samples were obtained. While blow counts ("N" value) could not be obtained, the soil samples and profiles were more visible than using a drilled boring approach. Additionally, unconfined compressive strengths using a hand penetrometer were obtained during the excavation on the matrix between the larger course non-cohesive components, which are a relevant property used similarly to "N" values.

As encountered within our subsurface test pits, the site was found to be mantled with topsoils, undocumented fills, and formational deposits. More specifically, these soils encountered within the explorations are described as follows and the exact depths of each test pit can be seen in Appendix B:

Topsoils:

We found topsoils to be six inches (6") deep in Test Pits 1, 2 & 3. Topsoils consist of brown sandy clay and silt with some roots and other organics, and classify as CL according to the Unified Soils Classification System. These soils are not suitable for the support of settlement sensitive structures.

Residual Soils:

Residual soils consisting of dark brown, firm to stiff, moist, sandy clays & clayey sands were found beneath the topsoil in Test Pit # 3. These soils classify as CL, SC and are of low plasticity with low to medium expansion index. These materials would usually be suitable for supporting of structures, but, the soils are different in consistency and settlement characteristics and should not be used for footing support where the remaining footings will be on Linda Vista Formation or Weathered Linda Vista Formation.

Weathered Formational Materials:

Weathered formational materials (a remnant of Linda Vista formation) were found at depths of one and one half feet to four feet (1.5'-4') in depth within the top two feet (2') of the Linda Vista formation. While very similar in characteristics to formational materials, they are significantly less cemented. They classify as CS/SC/SP/GP according to the Unified Soils Classification System, have a low potential for expansion and are very capable of supporting vertical loads.

Formational Materials:

Formational materials are Linda Vista formation. Formational materials classify as CS/SC/SP/GP (well cemented, orange –brown clayey sand and cobble conglomerate) according to the Unified Soils Classification System and, based on our experience with these materials, have a low potential for expansion and are capable of supporting high to very high loads.

See the boring logs in Appendix B for a more detailed presentation of the subsurface capability.

Geologic Reconnaissance

The site geology was reviewed by Mr. Michael Hart, a Certified Engineering Geologist in the State of California. The description of his site review, geologic conditions, and conclusions and recommendations are provided in Appendix E, "Preliminary Geologic Reconnaissance".

Based on Mr. Hart's information, it appears that no geologic hazards, such as active or potentially active faults, suspected landslides, or areas of potential soil liquefaction, exist at or within the immediate vicinity (within two hundred fifty feet [250'] of the site), and none were observed during the field evaluation. A potentially active fault associated with the Rose Canyon Fault exists approximately seven (7) miles to the east of this site as is further discussed in the Regional Faulting and Seismicity section of the Geologic Report, see Appendix E. The proximity to the Rose Canyon Fault is believed to be of little if any influence on the site and structures will be designed using the seismic coefficients as developed in the "Seismic Forces" and "Conclusions and Recommendations" sections of this report.

Existing Site Development

The site is currently developed with three (3) residential structures, one (1) on each lot. To the best of our knowledge, the structures are founded on continuous footings around the perimeter with individual pads inside. Both the footings and pads support a framed floor above the surface, thus creating a crawlspace between the floor and the original grades under the floor. To the best of our knowledge, the buildings are performing well with the foundations bearing on the upper Topsoils (weathered Linda Vista formation).

Site Drainage

The natural drainage of the site is primarily sheet flow to the north. An alley behind the three (3) lots collects all moisture to the south of the property, as 60th Street does to the east of the property; therefore, the only water that must be addressed is point fall moisture on the subject site.

Site Distress

A cursory visual observation of the exterior of the residences and associated improvements indicated that they have performed relatively well, with little sign of distress features/anomalies noted:

- Minor cracks were observed in the exterior stucco, located primarily around window and door openings.
- A variety of cracks and offsets were observed in the exterior concrete hardscape, suggesting some varying movement of the soils beneath the hardscape. This seemed more severe toward the south where the sandy clay was observed above the more granular material below.

LABORATORY TESTING

Laboratory tests were performed on the disturbed and undisturbed soil samples to determine their physical and mechanical properties, and their ability to perform appropriately under the demands of the project. The following tests were conducted on the sampled soils:

- Classification (ASTM D2487)
- Natural Moisture (ASTM D2216)
- Grain Size Finer Than #200 Sieve (-200)

- Expansion Index (UBC Standard 29-2)

A review of laboratory testing, including a description of the purpose and methodology of the tests, is provided, along with the quantitative and graphical (where applicable) test results (see Appendix C, "Laboratory Testing").

ANALYSIS AND DISCUSSION

General

In deriving recommendations for this project, the subsurface soils, proposed construction, and conditions of the existing structure and associated improvements were evaluated. Considerations were given to the potential for failure of the foundation soils, or the buildup of detrimental supplemental stress in the structural elements, due to differential vertical and/or lateral movement of the foundation soils.

Foundation Types

It is possible to have the load bearing components (foundations) supported on both formational and weathered formational material if they are similar in density. Non-bearing components (slabs) may be supported on the soils above, if the soils are similar to each other (even if not the Linda Vista formation), and non-expansive. Sometimes, this can be accomplished by supporting the slabs at the desired grade directly on the natural soils (or compacted fill) while the footings extend deeper to the desired and more competent bearing strata.

CONCLUSIONS AND RECOMMENDATIONS

General

Use of the quantitative results of laboratory test data, a thorough visual inspection of the soil types on the property during the excavation of the test pits, and our experience with similar soils and formational materials have aided in developing the conclusions and recommendations in this report.

In general, it is our opinion that the proposed improvements, as described, are feasible from a geotechnical standpoint, utilizing a shallow conventional foundation, if the recommendations of this report and generally accepted construction practices are adhered to. It is also our opinion that the site could be subjected to moderate to severe ground shaking in the event of a major earthquake along any of the faults mentioned previously, or other faults in the Southern California region; however, the seismic risk at this site is not significantly greater than that of the surrounding developed area (see "Seismic Forces" below).

We believe that the proposed development will have no negative consequences from geotechnical factors if the guidelines in this report are followed and other customary development techniques are used.

Recommendations are provided for each of the following areas of concern:

- Seismic Forces
- Foundations
    - Continuous Strip / Isolated Spread Footings
- Concrete Slabs-on-Grade
- Retaining Walls
- Earthwork
- Grading
- Surface Drainage
- Underdrain System
- Construction Observation

Seismic Forces

The following seismic design parameters should be used when developing loads and forces for structures:

Site Coordinates: 32.77232°N, 117.09668°W

S_S = 0.965 g S_{MS} = 1.075 g S_{DS} = 0.717 g S_1 = 0.369 g S_{M1} = 0.613 g S_{D1} = 0.409 g

Site Class: B from Table 1613A.5.2 based on

Occupancy Category: II from Table 1604.5

Therefore: Seismic Design Category: D

Foundations

The project and site are suited for the use of appropriate combinations of continuous strip footings, and isolated individual footings (pads), provided special care as described herein is exercised. Settlement should be within tolerable limits: One inch (1") total & three-quarters of an inch (¾") differential over a differential of fifteen feet (15').

Continuous Strip / Isolated Spread Footings

- All footings should be founded on competent well cemented Linda Vista formation or the equally dense less cemented weathered Linda Vista formation above. The depth to the competent material will range anywhere one and one half feet to four feet (1.5-4'') above Montezuma Road. Therefore, regardless of the slab elevation, if necessary, footings can be extended quite economically to the competent material below, without the need for any compaction or compaction testing for their support.

- Footings bearing a minimum of twelve inches (12") into competent material as described above, may be designed based on a maximum allowable soils pressure of four thousand (4,000) psf. Bearing values may be increased by twenty percent (20%) for each additional foot of depth, up to a maximum of one hundred and sixty percent (160%) of the designated values. Bearing values may be increased by thirty-three percent (33%) when considering wind, seismic, or other short duration live loads.

- To resist lateral forces, a lateral soil bearing pressure of one hundred and fifty (150) pcf may be used, with a coefficient of friction of three-hundredths (0.30) between the soil and concrete footings. Similarly, values of three hundred (300) pcf and 0.40 coefficient of friction may be used for the Linda Vista Formation.

- Footings shall be designed a minimum of fifteen inches (15") thick depending on the number of floors that are supported by that footing, i.e. if different floors are spanning different directions, the footing thickness should be based on the # of floors supported, not the # of floors in the building.

Floors Supported	Width
1	15 inches
2	15 inches
3	18 inches
4	24 inches

- All footings should be reinforced with a minimum of two (2) #4 bars at the top and two (2) #4 bars at the bottom (three inches [3"] above the ground). For footings (stem walls) over thirty inches (30") in height, additional reinforcement should consist of at least one (1) vertical #4 bar and one (1) longitudinal #4 bar, located at eighteen inches (18") o.c. vertically and horizontally in the center.

- All isolated spread footings should be designed utilizing the above given values and reinforced with #4 bars at twelve inches (12") o.c. in each direction (three inches [3"] above the bottom of the footing). Isolated spread footings should have a minimum horizontal dimension of twenty four inches (24").

- All loose soil found at the base of footings, when an excavation is opened, should be removed and the foundation extended to undisturbed competent soils as described, or founded on re-compacted over-excavated material.

- Our definition of "tolerable" limits of settlement should be confirmed by the Engineer or Architect of Record (EOR or AOR), based on the structural system proposed and all exposed finishes and if not acceptable, modifications to these recommendations (or designed parameters) should be made.

The preceding foundation recommendations are based on foundations bearing on suitable formational materials. None of the above is to preclude engineering requirements by the structural designer of the project, where calculations require more stringent measures. The above embedment and reinforcement considerations are minimum guidelines, which may be increased at the discretion of the engineer or designer responsible for structural considerations for the project.

Concrete Slabs-on-Grade

Concrete slabs on grade are suitable at this site if recommendations in this report are closely adhered to. The following recommendations are based on the assumption that slabs are placed on uniform soils. Slabs will be suitable if the following guidelines are closely adhered to:

- Most all soils on-site other than the organics which may be found in the top six inches (6") are suitable for the support of slabs.

- A uniform layer of four inches (4") of clean sand is recommended under any new slabs in order to more uniformly support the slab, help distribute loads to the soils beneath the slab, and act as a capillary break for upward migrating moisture. In addition, a plastic moisture barrier layer (6 mil) should be placed mid-height in the sand bed to act as a vapor barrier.

- Concrete slabs-on-grade should have a nominal thickness of five inches (5") and should be reinforced with #4 bars placed at mid-depth in the slab at twelve inches (12") on center in each direction.

- Adequate control joints should be installed to control the unavoidable cracking of concrete that takes place when undergoing its natural shrinkage during curing. The control joints should be well located to direct unavoidable slab cracking to areas that are desirable by the designer.

- If unsuitable fills are found on the south sides of the lots, it is possible that they will have to be removed, and some of the better excavated material be used as fill to establish a more uniform support of slabs.

The amount of shrinkage will depend on the water cement ratio of the concrete. Therefore, all slab concrete should have a minimum water cement ratio. The inclusion of additives such as water reducers or super plasticizers is recommended for the ease in placement of the concrete, as well as improved future performance. Control joints should be spaced closer if the water content is higher; these recommendations should be provided by the Engineer of Record.

The aforementioned precautions will not prevent slab movement if the underlying soils become moistened; however, they will minimize the damage if such movement occurs.

Retaining Walls

We understand that retaining walls will be required around the perimeter of the building to support the soils above the subterranean parking garage. Since the elevation of the garage slab is unknown, as well as the elevation of the perimeter grades around the building, the wall heights are unknown; however, the values and recommendations provided herein are assuming that the walls are anywhere between three feet and eight feet (3’-8’) in height.

The walls should also be designed and constructed in accordance with the following recommendations:

- Unrestrained cantilever retaining walls should be designed using an active equivalent fluid pressure of forty (40) pcf. This assumes that the on-site materials will be used, as well as any granular import. This also assumed that the backfill is level. If the sloping backfill is contemplated, the following values should be used:

Condition	3:1 Slope	2:1 Slope
Active	45 pcf	50 pcf

- Any other surcharge loadings within the 1:1 slope extending from the surface to the base of the wall should be analyzed in addition to the above values.

- Due to the Seismic Design Category D, an additional (increased) lateral pressure on the retaining walls due to earthquake motions must be included. An increase in soil pressure equal to twenty percent (20%) for Seismic Design Category “D,” of the above provided active pressure values should be added to the walls for inertial forces due to seismic activity when designing retaining walls. All applicable increases in allowable stresses and/or other coefficients, as well as load duration factor reductions, may be utilized when including this short duration loading on retaining walls.

- Retaining wall backfill should be placed and compacted in accordance with the "Earthwork" section and Appendix D, "Grading Specifications." Where there is a conflict, the more stringent recommendations shall be used unless otherwise specified by the engineer.

- If the tops of retaining walls are restrained from movement (i.e. a concrete podium deck), they should be designed using an additional uniform soil pressure of 5 x H psf, where H is the height of the fill in feet from the ground level to top of footing. If desired, an at-rest equivalent fluid pressure of sixty (60) pcf may be used instead of the 5 x H.

- Similar to the regular foundations, passive soil resistance may be calculated using an equivalent fluid pressure of one hundred fifty (150) pcf for the weathered formational material, and three hundred (300) pcf, for formational materials for retaining walls. This value assumes that the formational material being utilized to resist passive pressures extends horizontally two (2) times the depth of the fill.

- A coefficient of friction of 0.40 between the formational material and concrete footings may be utilized to resist lateral loads in addition to the passive soil pressures above.

- Retaining walls should be braced and monitored during compaction of backfill. If this cannot be accomplished, the compactive effort should be included as a surcharge load when designing the wall. The inclusion of the seismic force should account for this.

- All walls should be provided with adequate back drainage to relieve hydrostatic pressure in accordance with Appendix D, Figure D-6 "Site Retaining Wall Drainage." All exterior site retaining walls should, at a minimum, have the strike mortar omitted in the lowest course to allow for drainage.

- Retaining walls that are designed as unrestrained should be backfilled prior to constructing any floor attached to the top of the wall.

Earthwork

Earthwork should be performed in accordance with pertinent city standards, Appendix D, "Grading Specifications," and the following recommendations:

- **Commencement/Removals:**

  Prior to grading, areas of proposed improvement should be cleared of surface and subsurface debris, and stripped of vegetation and organic top soil. All of this material must be removed from the site.

- **Excavations:**

  Some of the existing fill soils found to mantle the site in the exploratory excavations (south side) may not be desirable for the support of structures or settlement-sensitive improvements in their present state. Where settlement-sensitive improvements are to be constructed (including interior and exterior slabs), the unsuitable soils should be removed and replaced with properly compacted fill. This should be performed to a distance of five feet (5') beyond any proposed hardscape surfaces outside of the building.

  While refusal was encountered during the geotechnical investigation, the equipment was limited in size due to restrictions in the access when considering the range of proposed slabs and foundations. We believe that there will be no issue requiring special equipment or blasting. A large excavator and/or a front end loader should be suitable for all excavation.

  Cut slopes should remain stable for a short period of time, if limited to six feet (6') in height and slopes not exceeding 1.5:1 (horizontal to vertical) in soils and 1:1 (horizontal to vertical) in formational material. Care should be exercised when making removals adjacent to, or near, existing foundations that will remain during construction. Shoring will be required when any surcharge loadings from existing structures fall within a 1:1 line extending from the top of any footings to the bottom of the surcharge loading (see Figure 4).

  No equipment, material, soil stockpile, other loads, or surcharge should be placed at the top of slopes within a horizontal distance from the top of the slope equal to one-half (1/2) the height of the excavation. Our office should be contacted to observe all temporary slopes during construction to determine if any adverse geologic conditions are exposed which would affect the stability of the slope.

- **Fills:**

  After excavating is completed, areas to receive fill and/or structural improvements should be scarified to a minimum depth of eight inches (8"), brought to near optimum moisture content, and properly re-compacted to at least ninety percent (90%) relative compaction (based on ASTM D1557). All fill slopes should be properly compacted to ninety percent (90%) relative compaction in order to avoid erosion and slough age. A minimum overall slope of 2:1 (horizontal to vertical) should be maintained. When fills are required to support any area of a slab, then the entire slab should be supported by a minimum of eighteen inches (18") of fill to avoid differential settlement.

  Fills should generally be placed in lifts not exceeding eight inches (8") in thickness. If importing soil is planned, soils should be non-expansive and free of debris and organic matter. Prior to importing, soils should be visually observed, sampled, and tested at the borrow pit area to evaluate soil suitability as fill.

Grading

All areas that require increased grades shall be filled with soil compacted to 90%. All areas that previously received unsuitable fills, should be removed down to natural soils beneath the fills, the base of the area scarified and compacted, adequately benched, and the soils placed back in lifts not exceeding eight inches (8”) in thickness and compacted to a minimum density of ninety percent (90%) Modified Proctor. No permanent fill slopes shall be steeper than 2:1 (horizontal to vertical) and no permanent cut slopes shall be steeper than 1.5:1 (horizontal to vertical).

All recommendation in Appendix D should be adhered to while conducting the above work. Care should be taken when approaching existing structures, so as not to damage the structures. Some shoring may be required adjacent to any adjacent structures in order to avoid undermining any foundations.

Surface Drainage

Adequate drainage precautions at any site are important. Under no circumstances should water be allowed to pond against or adjacent to footings, foundation walls, or retaining walls. The ground surface surrounding the building should be relatively impervious in nature, and slope to drain away from the building in all directions, with a minimum slope of five percent (5%) for a horizontal distance of ten feet (10’). Area drains or surface swales should then be provided to accommodate runoff and avoid any ponding of water. Roof gutters and downspouts with tightline drains should be installed on the proposed structure, and discharged to flow to suitable outlets a minimum of ten feet (10’) away from the foundation. Surface and area drains should not be connected to any wall drainage or underdrain system. Drainage should also be diverted away from the top of slopes to avoid erosion and “creep.” Surfaces should be adequately vegetated or otherwise covered with hardscape surfaces and provided with appropriate energy dissipaters, where applicable, to avoid pending erosion.

All the water from the roof and other hardscape surfaces on the site must be discharged out to one (1) of the streets. Roof gutters and downspouts are absolutely necessary to collect the water and discharge it to the appropriate outlet. If sheet flow on the surface cannot maintain a minimum slope of ¼” in ten feet (10’), then area drains must be installed to collect the water and run it to a low point at 60th Street or Montezuma, most presumably at the northwest corner of this property.

Underdrain System

As we understand, there will be retaining walls around the perimeter of the garage. Perimeter drains should be installed below the slab level of the garage. These drains should consist of a four inch 4” perforated plastic pipe surrounded in gravel, in turn surrounded by filter fabric (see Figure D7). Since it is not known what the elevation of the slab is, it is not known if positive drainage to the street is possible. Should direct drainage to the street not be possible, the pipes around the perimeter should still be constructed below the slab level; however, they should be directed to a sump where a pump can be utilized to discharge the water to either Montezuma or 63rd Street. A generator backup should be seriously considered, so that during periods of extreme weather (with possible power outages), detrimental water can still be removed from behind the retaining walls to avoid flooding any subterranean spaces. The drain should be constructed to easily receive and discharge water that contacts the retaining wall. Miradrain 6100N is a very good product to aid in waterproofing the wall and provide an unrestrictive pathway to the drain (see Figure D-7).

Waterproofing

Waterproofing should be utilized on all exterior walls that will be below the exterior grades. The waterproofing should be adequate to resist moisture infiltration into the walls. The product to be used shall be specified and approved by the Engineer or Architect of Record. The drain must be of adequate characteristics or be covered to avoid damage during backfill. Miradrain 6100N is an excellent product to accomplish this, as well as assisting in directing water to an underdrain system (see Figure D-7).

Construction Observation

The following services should be conducted under the direction and supervision of a qualified geotechnical engineer prior to or during construction of the proposed improvements (if applicable):

1. Grading and foundation plan review, prior to grading.
2. Observation of all subgrade preparation, subsequent to any removals and prior to any fill or concrete placement.
3. Observation and testing of any fill placement over two feet (2’) in depth and preparation of compaction report (see Appendix D)
4. Observation of foundation excavations.
5. Observation of any conditions that vary from the conditions as described within this report.

It is possible that jurisdictional agencies may require additional services for documentation during construction, where applicable. These requirements may include the review or observation of one (1) or more of the following:

- Exposed undercuts
- Reinforcing placement in slabs
- Reinforcing placement in footings
- Waterproofing

- Subdrain installation
- Area drain installations
- Finish rough grade of slopes and area drain

The owner/builder, Architect, Engineer of Record and Contractor should consult the governing agencies to determine the extent of their requirements prior to commencing work, to avoid costly delays during construction, and to have the required services included in the construction budget for the project.

MISCELLANEOUS

General

Building sites, in general, in particular, need maintenance to continue to function and retain their value. Many homeowners are unaware of this and allow deterioration of their property. It is important to familiarize homeowners (property managers) with some guidelines for maintenance of their properties and make them aware of the importance of maintenance.

Even when the original drainage for a property is excellent to address potential hazards associated with drain water, it is the buyer's responsibility to **maintain** these safety features by observing a prudent program of lot care and maintenance. Failure to make regular inspection and maintenance of drainage devices and sloping areas may cause severe financial loss. In addition to his/her own property damage, he/she may also be subject to civil liability for damage occurring to neighboring properties as a result of his/her negligence.

Maintenance Guidelines for Property Managers and HOAs

The following maintenance guidelines are provided for the protection of the homeowner’s investment:

- Surface drainage must be directed away from structural foundations to prevent ponding of storm waters or irrigation adjacent to footings.
- Care should be taken that slopes, terraces, berms (ridges at crown of slopes) and proper lot drainage is not disturbed. Surface drainage should be conducted from the rear yard to the street through the side yard, or to natural drainage ways within the property boundary.
- In general, roof and yard runoff should be conducted to the street using non-erosive devices such as sidewalks, drainage pipes, ground gutters, and driveways. Drainage systems should not be altered without expert consultation.

Limitations

It must be noted that no structure or slab should be expected to remain totally free of cracks and minor signs of cosmetic distress. The flexible nature of wood and steel structures allows them to respond to movements, resulting from minor unavoidable settlement of fill or natural soils, the swelling of clay soils, or the motions induced from seismic activity. In addition, products containing cement also shrink during natural curing. All of the above can induce stresses that frequently result in cosmetic cracking of wall and floor surfaces, such as stucco or interior plaster, floor tiles, or other brittle finishes. This is especially true when considering an addition or modification to an existing building.

Data for this report was derived from surface observations at the site, knowledge of local conditions, and a visual observation of the soils exposed in the subsurface excavations. The recommendations in this report are based on our experience in conjunction with the limited soils exposed at this site and neighboring sites. We believe that this information gives an acceptable degree of reliability for anticipating the behavior of the proposed improvements; however, our recommendations are professional opinions and cannot control nature, <u>nor can they assure the soil profiles beneath or adjacent to those observed</u>; therefore, no warranties of the accuracy of these recommendations, beyond the limits of the obtained data, is herein expressed or implied. This report is based on the evaluation at the described site and on the specific anticipated construction as stated herein. If either of these conditions is changed, the results would also most likely change.

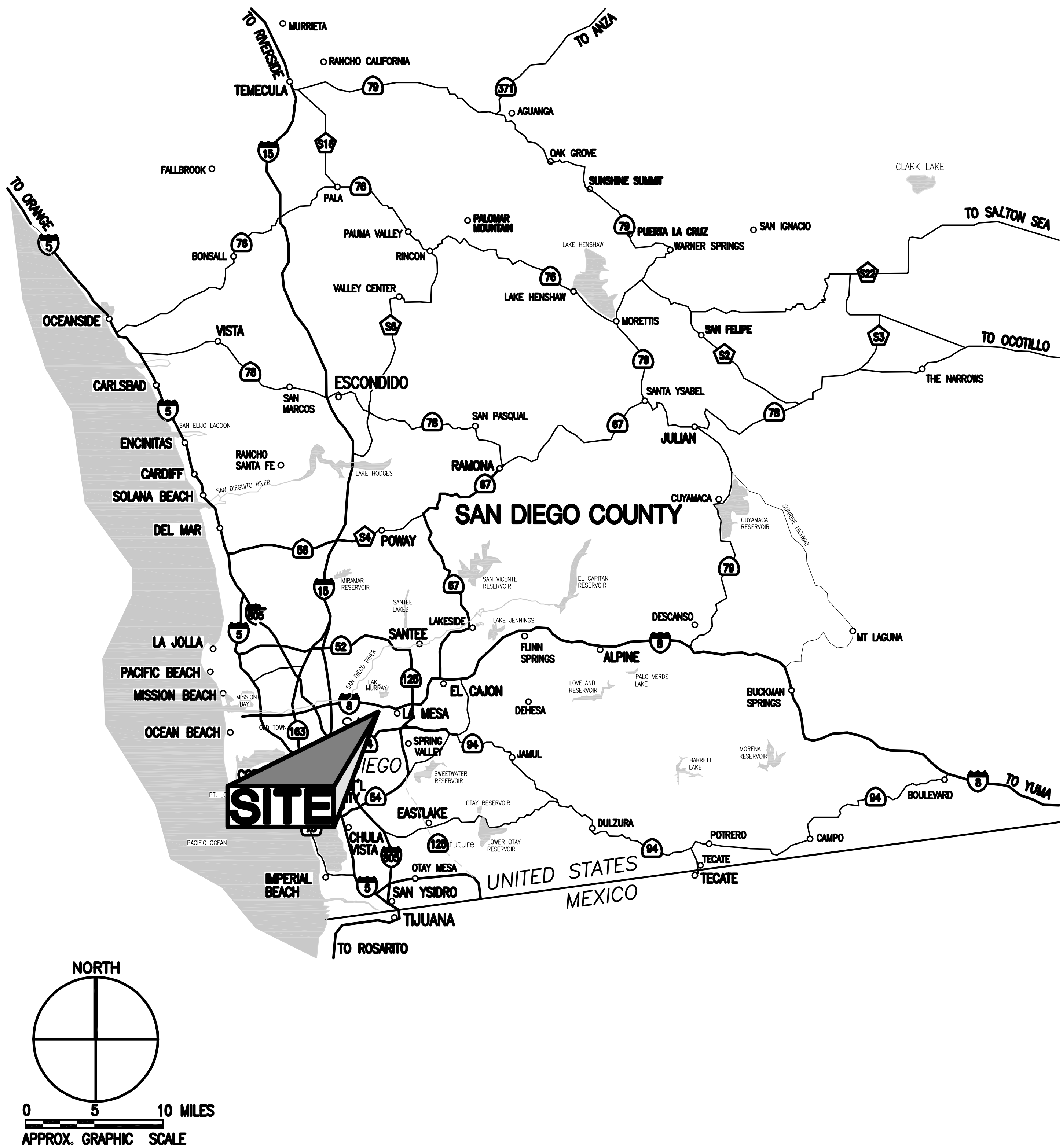
Man-made or natural changes in the conditions of a property can occur over a period of time. In addition, changes in requirements due to state-of-the-art knowledge and/or legislation are rapidly occurring. As a result, the findings of this report may become invalid due to these changes; therefore, this report for the specific site is subject to review and not considered valid after a period of one (1) year, or if conditions as stated above are altered. This report is not meant to imply nor does it offer any warranty whatsoever as to the future performance or value of the property. Use of this report is for the sole purpose of the client. It is understood that Accutech Engineering Systems, Inc. will be compensated in full for any costs of litigation that may arise from the use of this report, including, but not limited to, fees for staff, attorneys, and/or expert witness testimony.

It is the responsibility of the owner or his representative to insure that the information in this report be incorporated into the plans and/or specifications and construction of the project. It is advisable that a contractor familiar with construction details typically used to deal with the local subsoil and seismic conditions be retained to build the structure.

We hope the report provides you with necessary information to continue with the development of the project. If you have any questions regarding this report, or if we can be of further service, please do not hesitate to contact us at 619.261.2619.

Very truly yours,
ACCUTECH ENGINEERING SYSTEMS, INC. Robert J. Randall RCE #707 RJR: dm

REGISTERED PROFESSIONAL ENGINEER
ROBERT JOHN RANDALL
No. GE 707
Exp. 3-31-19
GEOTECHNICAL
STATE OF CALIFORNIA



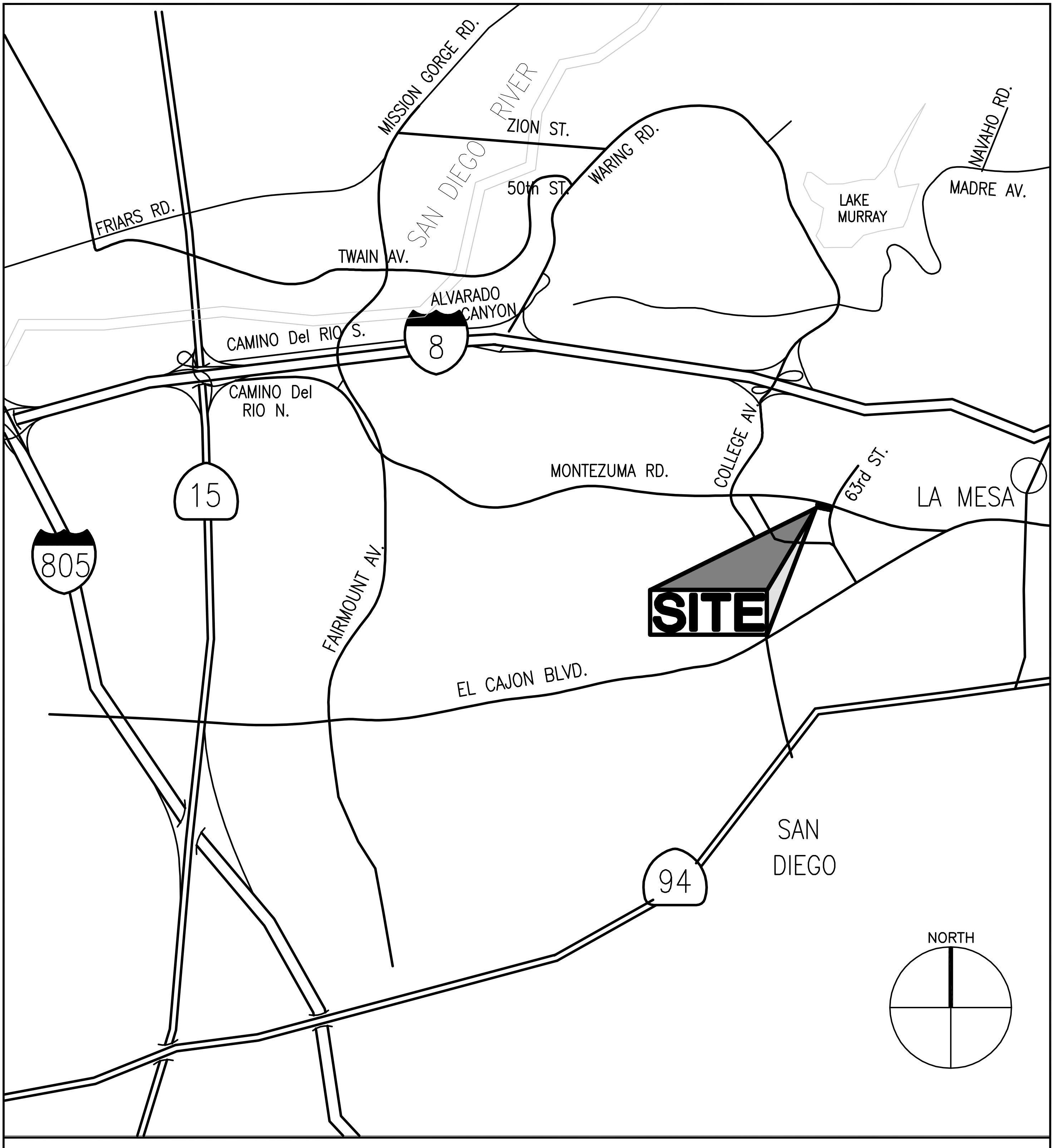
SITE VICINITY MAP

ACCUTECH ENGINEERING

REGISTERED PROFESSIONAL ENGINEER
ROBERT JOHN RANDALL
C 040385
Exp 3/31/19
CIVIL
STATE OF CALIFORNIA

REMAX Pacific
6253-6265-6275 MONTEZUMA RD.
SAN DIEGO, CA 92115

DATE:	DRWN BY:	FIGURE NO.:	PROJECT NO.:
07-26-2018	GLN	1	18423-1



SITE LOCATION MAP

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Exp 3/31/19
CIVIL
STATE OF CALIFORNIA

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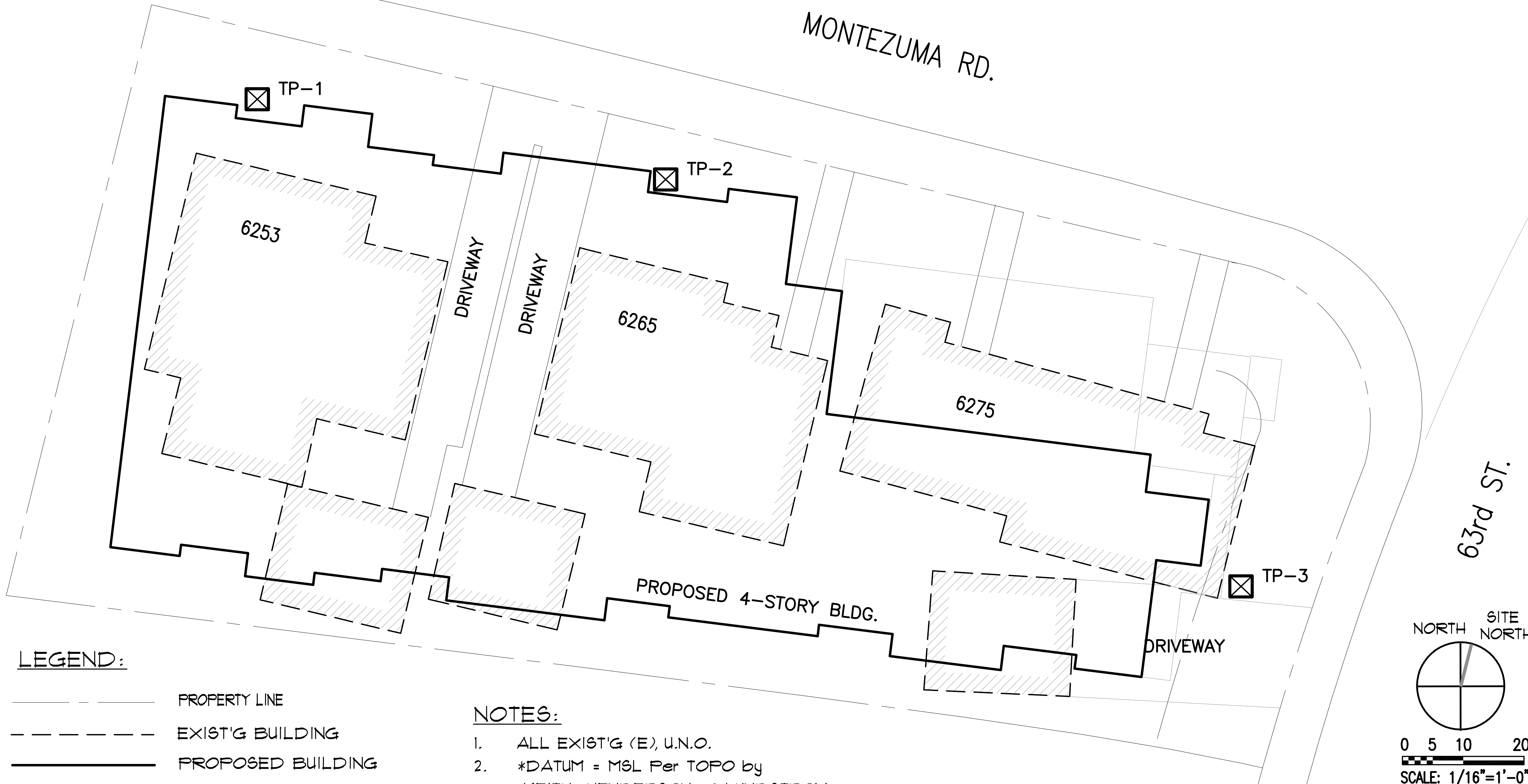
DATE:	DRWN BY:	FIGURE NO.:	PROJECT NO.:
07-26-2018	GLN	2	18423-1



LEGEND:

- PROPERTY LINE
- EXIST'G BUILDING
- PROPOSED BUILDING
- EXIST'G HARDSCAPE
- 463.36 — *BENCHMARK = BRASS PLUG TOP of CURB WEST SIDE 67th ST.
- TP-x — LOCATION AND NUMBER of EXPLORATORY TEST PIT

NOTES:

1. ALL EXIST'G (E), U.N.O.
2. *DATUM = MSL Per TOPO by KEITH HENDERSON of LUNDSTROM ENG'R'G and SURVEY'G, INC., DATED 03-07-18
3. ALL ELEV's SHOWN ARE APPROX.

SITE PLAN/LOCATION of TEST PITS

ACCUTECH ENGINEERING

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CIVIL
STATE OF CALIFORNIA

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SAN DIEGO, CA 92115

DATE:	DRAWN BY:	FIGURE NO.:	PROJECT NO.:
07-26-2018	GLN	3	18423-1

Design Maps Summary Report

User–Specified Input

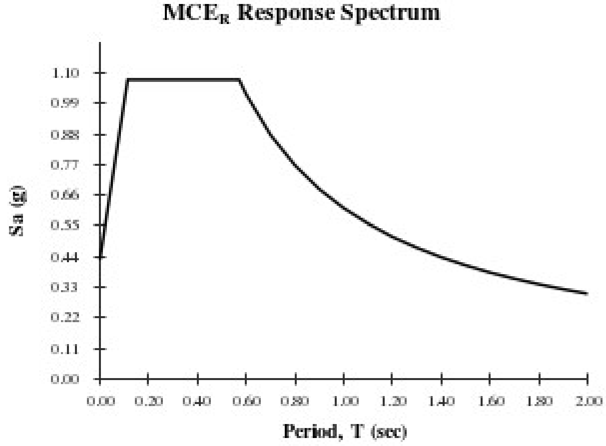
Report Title	18423-1 6253-6265-6275 Montezuma Road Mon July 30, 2018 21:14:59 UTC
Building Code Reference Document	2012/2015 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	32.77232°N, 117.09668°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

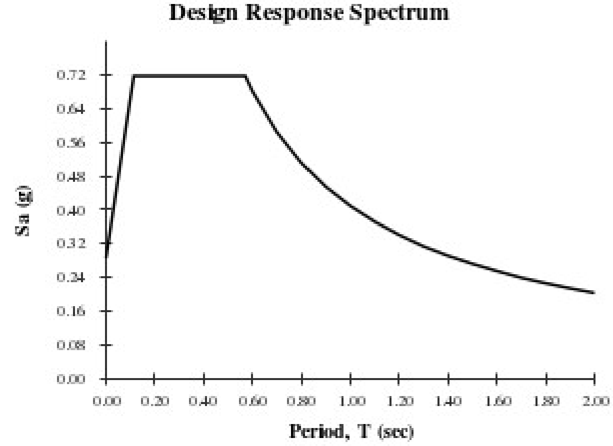
S_S =	0.965 g	S_{MS} =	1.075 g	S_{DS} =	0.717 g
S_1 =	0.369 g	S_{M1} =	0.613 g	S_{D1} =	0.409 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



APPENDIX A

REFERENCES

REFERENCES

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2. Department of Public Works, County of San Diego, California, 1977, *Faults and Epicenters,* Scale of 1 inch:2 miles.

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4. Harris, M.E., 1987, *Geology of San Diego County, California, A Bibliography with Subject Index*, California Division of Mines and Geology, Special Publication 96.

5. Jennings, C.W., 1977, *Geologic Map of California*, California Division of Mines and Geology, Scale 1:750,000.

6. Jennings, C.W., 1992, *Preliminary Fault Map of California*, California Division of Mines and Geology, Revision of the 1975 Fault Map of California, Scale 1:750,000, Open File Report 92-03.

7. Kennedy, M.P., and Peterson, G.L., 1975, *Geology of the San Diego Metropolitan Area, California*, California Division of Mines and Geology, Bulletin 200.

8. Kennedy, M.P., Tan, S.S., Chapman, R.H. and Chase, G.W., 1975, *Character and Recency of Faulting, San Diego Metropolitan Area, California*, California Division of Mines and Geology, Special Report 123.

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10. City of San Diego, 1995, San Diego Seismic Safety Study.

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14. *Landslide Hazard Identification Program*, California Division of Mines and Geology, Annual Report 1990.

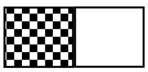
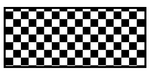
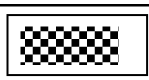
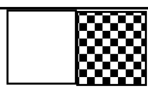
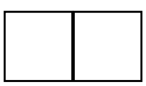
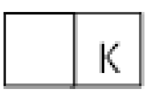
15. Seed, H.B. and Idriss, I.M., 1982, *Ground Motions and Soil Liquefaction During Earthquakes*, Earthquake Engineering Research Institute.

APPENDIX B

Subsurface Exploration

TABLE OF CONTENTS

LEGEND

Symbol	Description
∇	Groundwater level or groundwater seepage at the time of drilling could vary seasonally.
	Location of Undisturbed sample taken in a boring using a 2 3/8 inch I.D. modified California Split Tube Sampler liner rings.
	Location of Undisturbed sample taken using a 3 inch O.D. thin-walled tube sampler (Shelby Tube) hydraulically pushed.
	Location of disturbed sample taken in a boring using a standard tube sampler, 2 inch O.D. 1 3/8 inch I.D. See Blow Count.
	Location of bulk disturbed sample taken from auger cuttings in borings or shovel in test pits.
	Location of Undisturbed sample taken in a test pit using a 2 3/8 inch I.D. "California" liner ring and hand drive adapter.
	Location of carved, chuck or block Undisturbed sample in a test pit.
D	Sample disturbed during sampling. No recovery.
	Sample obtained using a 3400 lb. “Kelly bar” free falling (12”)
Blow Count	Number of drives of sampling device for 6 inch sample, unless noted otherwise. For example: 14/13/12 = (4) — 14 blows of a 140 lb. weight free falling (30”) were required to drive the sampling device the first 6 inches then 13 blows for the next 6”, etc. 50(4) — 50 blows of the weight were required the drive the sampling device 4 inches. $\frac{18}{14}$ = — Blow count converted to SPT when other samplers are used. See attached "Blow Count Conversion".

DEFINITION OF TERMS

Term	Definition
ϕ	Angle of internal friction (degrees)
-200	Material passing the #200 sieve (%)
App Dnsty	Apparent Density is the estimated density of the soil, at the depth noted, during field observation and classification (pounds per cubic foot).
App Moist	Apparent Moisture is the estimated moisture content, at the depth noted, during field observation and classification (%).
Cf	Coefficient of Friction
DD	Dry Density
EI	Expansion Index
HD	Hand Drive Sample
HP	Unconfined compressive strength (hand penetrometer, tsf)
ID	Inside Diameter
KSF	Kips per square foot
LL	Liquid Limit (%)
MC	Natural Moisture Content
MSL	Mean sea level
NP	Non-Plastic
OD	Outside Diameter
PI	Plastic Index (%)
PL	Plastic Limit (%)
PSF	Pounds per square foot
SPT	Standard Penetration Test
TSF	Tons per square foot
UC	Unconfined compressive strength (cohesion intercept, ksf)
USCS	Unified Soil Classification System
WD	Wet Density

BLOW COUNT CONVERSION (N-VALUE)

The blow count representation of the penetration resistance of a soil (N-Value) is achieved by driving a standard 2 inch O.D. split-barrel sampler utilizing a drive weight of 140 pounds impacting the sampler from a fall of 30 inches. This method is known as the Standard Penetration Test (SPT) and is also used to obtain disturbed samples. Frequently, a larger sampler with brass rings is used to obtain undisturbed samples. A correlation between SPT blow count and blow count from a larger diameter ring lined sampler used may be obtained by considering drive energy created by the fall of the 140 pound weight over the effective cross sectional area of the samplers. The drive energy of a larger 3 inch diameter sampler (133 ft-lb/in^2) divided by the drive energy of the standard 2 inch diameter sampler (211 ft-lb/in^2) results in a conversion factor of 0.630. The blow count of the 3 inch diameter sampler may be multiplied by this conversion factor to equate it to SPT blow count.

Correlation of blow count between SPT and ring lined split-barrel drive sampler:

Given: Standard drop hammer weight of 140 pound drop of 30 inches

O.D. SPT 2 in.	O.D. Split-barrel 3 in.
I.D. SPT 1.375 in.	I.D. Split-barrel 2.375 in.

Effective Area of SPT:

$A = \pi d^2/4$

$A = \pi(2\text{ in})^2/4 - \pi(1.375\text{ in})^2/4$

Effective Area = 1.657 in^2

Drive Energy SPT:

(140 lb)(2.5 ft)/1.657 in^2

$\underline{\underline{211\text{ ft-lb/in}^2}}$

Effective Area of Ring Lined Split-Barrel Sampler:

$A = \pi d^2/4$

$A = \pi(3\text{ in})^2/4 - \pi(2.375\text{ in})^2/4$

Effective Area = 2.638 in^2

Drive Energy Ring Lined Split-Barrel Sampler:

(140 lb)(2.5 ft)/2.368 in^2

$\underline{\underline{133\text{ ft-lb/in}^2}}$

$\therefore$ Conversion (C):

$C = 133\text{ ft-lb/in}^2 \div 211\text{ ft-lb/in}^2$

$C = 0.630$

TEST PIT LOG TP-1

Equipment: Backhoe w/ 18" Bucket	**Type:** Test Pit **Dimensions:** 20" x 6" x 3'	**Date Logged:** 7/27/18
Hole Elevation: 103 **Datum:** Street = 100	**Groundwater Depth:** NA	**Logged By:** RJR

Depth (ft)	**Location:** Northwest corner of lots		**Field Information**				**Laboratory**	**Misc.**
	USCS	Field Description and Classification	Sample Type	H P	Apparent Density (pcf)	Apparent Moisture (%)		
0–1	CL OL	Brown, sandy clay and silt, dry Some roots and other organics **TOPSOIL**		10				
1–2	CS SC SP GP	Sand with clay to clay with sand and cobbles, becoming sandier. Slightly moist. Dense, rust & olive color Cemented **WEATHERED FORMATION**	▩	7.5	125			
2–5	CS SC SP GP	Clay chunk lenses, very moist Orange-brown, clayey sand and cobble conglomerate Well-cemented Slightly moist, dense Hard digging **LINDA VISTA FORMATION**	▩	10.0+	135		M.C. = 9.7 - 200 = 26.5	
5–10		Refusal in cobbles: Bottom of test pit @ 5'-0"						

Project Name: 6253-6265-6275 Montezuma Road	**Project #:** 18423-1
Project Location: 6253-6265-6275 Montezuma Road, San Diego, CA 92115	**Figure #:** B-4

TEST PIT LOG TP-**2**

Equipment: Backhoe w/ 18" Bucket	**Type:** Test Pit **Dimensions:** 20" x 6" x 5'	**Date Logged:** 7/27/18
Hole Elevation: 103 **Datum:** Street = 100	**Groundwater Depth:** NA	**Logged By:** RJR

Depth (ft)	**Location:** Center of lots		**Field Information**				**Laboratory**	**Misc.**
	USCS	Field Description and Classification	Sample Type	H P	Apparent Density (pcf)	Apparent Moisture (%)		
0–1	CL OL	Brown, sandy clay and silt, dry Some roots and other organics **TOPSOIL**						
1–2.5	CS SC SP GP	Sand with clay to clay with sand and cobbles, becoming sandier. Slightly moist. Dense, rust & olive color Cemented **WEATHERED FORMATION**		8.5				
2.5–5	CS SC SP GP	Note: Weathered formation transitioning into Linda Vista formation @ 2-3' Clay chunk lenses, very moist Orange-brown, clayey sand and cobble conglomerate Well-cemented Slightly moist, dense Hard digging **LINDA VISTA FORMATION**		10.0+				
5–10		Refusal in cobbles: Bottom of test pit @ 5'-0"						

Project Name: 6253-6265-6275 Montezuma Road	**Project #:** 18423-1
Project Location: 6253-6265-6275 Montezuma Road, San Diego, CA 92115	**Figure #:** B-5

TEST PIT LOG TP-3

Equipment: Backhoe w/ 18" Bucket	**Type:** Test Pit **Dimensions:** 20" x 6" x 4'	**Date Logged:** 7/27/18
Hole Elevation: 104 **Datum:** Street = 100	**Groundwater Depth:** NA	**Logged By:** RJR

Location: Southeast corner of lots

Depth (ft)	USCS	Field Description and Classification	Sample Type	H P	Apparent Density (pcf)	Apparent Moisture (%)	Laboratory	Misc.
0–0.5	CL OL	Brown, sandy clay and solt, dry Some roots and other organics **TOPSOIL**						
0.5–2	CL SC	Stiff, brown Some roots **RESIDUAL**	▩	2.0 7.0	110 125	15	EI = 53 MC = 16.2 - 200 = 53	
2–4	CS SC SP GP	Note: Weathered formation transitioning into Linda Vista formation @ 2-3' Clay chunk lenses, very moist Orange-brown, clayey sand and cobble conglomerate Well-cemented Slightly moist, Hard digging **LINDA VISTA FORMATION**	▩	10.0+	135		MC = 6.8 -200 = 18.3	
4		Refusal in cobbles: Bottom of test pit @ 4'-0"						

Project Name: 6253-6265-6275 Montezuma Road	**Project #:** 18423-1
Project Location: 6253-6265-6275 Montezuma Road, San Diego, CA 92115	**Figure #:** B-6

APPENDIX C

LABORATORY TESTING

LABORATORY TESTING

Laboratory tests were performed in general accordance with the accepted practice of the American Society for Testing and Materials (ASTM), the Uniform Building Code (UBC), and other suggested methods. A brief description of the tests performed is as follows:

- **CLASSIFICATION** - Field classifications are prepared in the field and are verified in the laboratory by a visual examination per (ASTM D2487). Further classification is provided with the aid of supplemental laboratory testing of selected samples obtained in the field. Samples are classified, as coarse or fine grained, well or poorly graded, high or low plasticity, per the Unified Classification System.

- **NATURAL MOISTURE & DENSITY** - Moisture contents and dry densities are determined for representative soil samples in accordance with ASTM D2216. This information is an aid to classification and assists in recognition of variations in material consistency with depth. The dry unit weight is determined in pounds per cubic foot, and the in-situ moisture content is determined as a percentage of the dry unit weight. The results are summarized in the excavation and/or boring logs and the summary of laboratory testing within this section of the report.

- **ATTERBERG LIMITS** - The plastic and liquid limits and the plasticity index are determined in accordance with ASTM D4318. This test is performed on the portions of the sample passing the #40 sieve, and assists in classifying the fine grained soils into low or high plasticity fines.

- **BULK DENSITY** - The density of materials by the water displacement method is used for determining the bulk density of an irregular shaped soils or rock sample. By weighing a sample in both a natural state and submerged state, the buoyant force is obtained. This equates to the volume of displaced water, and thus the volume of the sample. The sample is coated with wax to keep water from seeping into it. Calculations are performed to compensate for the weight of the wax, and bulk densities (pounds per cubic foot) of the samples are obtained.

- **GRAIN SIZE FINER THAN #200 SIEVE (-200)** - Samples are washed through a #200 sieve (opening size of 0.7mm) in accordance with ASTM D1140. The weight of the dried material passing the #200 sieve is represented as a percentage of the total sample dry weight. The results are presented on the gradation test results sheets, within this section of the report.

- **GRAIN SIZE DISTRIBUTION** - The grain size distribution is determined for representative samples of the native soils in accordance with ASTM D422. Samples are washed through a #200 sieve (opening size of 0.7mm) and then mechanically vibrated through a series of sieves of various size openings. The results are presented on the gradation test results sheets, within this section of the report.

- **OPTIMUM MOISTURE / DENSITY** - The maximum dry density and optimum moisture content of typical soils are determined in the laboratory in accordance with ASTM D1557, Methods A and/or C. Method A specifies that a four (4) inch diameter cylindrical mold of 1/30 cubic foot of volume be used for soils. Method C specifies that a six (6) inch diameter cylindrical mold of 1/13 cubic foot of volume be used for soils. Moisture Content of the soil sample is varied to determine the "Optimum Moisture Content" at which the "Maximum Density" occurs. The results of these tests are used in conjunction with the field density tests to determine the degree of compaction of the fill and/or native soils.

- **EXPANSION INDEX** - Expansion Index tests on remolded samples are performed on representative samples of soils per UBC Standard 29-2. The test is performed on the portion of the sample passing the #4 standard sieve. The sample and is then compacted in a 4-inch-diameter mold at a saturation of approximately 50 percent. The specimen is placed in a consolidometer with porous stones at the top and bottom, subjected to a total normal load of 12.63 pounds (144.7 psf), and the sample is allowed to consolidate for a period of 10 minutes. The sample is submerged in water and the change in vertical movement is measured and recorded until the rate of expansion becomes nominal. The expansion index is reported as the total vertical displacement in inches times 1000.

- **DIRECT SHEAR** - Direct Shear tests are performed in accordance with ASTM D3080, to determine the failure envelope relating confining pressure to shear strength, based on yield shear strength. This is given as "0", the angle of internal friction and "C" the unconfined strength (cohesion intercept). A minimum of three (3) samples are tested at different vertical loads. The shear stress is applied at a constant rate of strain at approximately 0.05 inch per minute and the ratio is thus obtained and plotted.

- **RESIDUAL DIRECT SHEAR** - Residual shear samples are sheared, as describe in the preceding paragraph, with a greatly reduced shearing rate. The upper portion of the specimen is pulled back to the original position and the shearing process is repeated until no further decrease in shear strength is observed with continued shearing (at least 3 times). There are two methods to obtain the shear values: (a) the shearing process is repeated for each load applied and the shear value for each normal load is recorded. One or more specimens can be used in this method; (b) only one specimen is needed and a very high normal load (approximately 9,000 psf) is applied from the beginning of the shearing process. After the equilibrium state is reached (after "relaxed"), the shear value for that normal load is recorded. The normal loads are then reduced gradually prior to re-shearing. The shear values are recorded for different normal loads after they are reduced and the sample is "relaxed". This test is of value in areas of known hillside failures and for hillside stability repair.

LABORATORY TEST RESULTS SUMMARY SHEET

SAMPLE		DD %	MC %	-200	PL	PI	EI	Classification	Misc./Comments
Loc.	Depth								
TP-1	3’		9.7	26.5				CS/SC/SP/GP	
TP-3	2’		16.2	53			53	CL/SC	
TP-3	4’		6.8	18.3				CS/SC/SP/GP	

- **SWELL/CONSOLIDATION** - A natural undisturbed, or re-molded sample is used to determine the potential for expansion or consolidation under anticipated loading conditions when the sample is subjected to increased water content. This test is run in accordance with ASTM D2435 and D4546 and performed on a single counter balance lever system type consolidometer. Samples are confined within a ring and loaded vertically with pressures similar to those anticipated under expected real life conditions.

  A seating cycle is applied to compensate for disturbances to the sample during transport. The sample is loaded to approximately one-half (1/2) over burden pressure by doubling successive loads. Each loading condition is placed for one (1) hour until approximately one-half (1/2) over burden pressure, then the sample is rebounded to start the consolidation test.

  The sample is then allowed to consolidate once again under varying load conditions, until movement is nominal at each load, and the final reading is recorded for each load. The load amount is increased until the anticipated load is reached, at which time the sample is submerged in water, and allowed to consolidate further or expand when subjected to this increased moisture until movement is once again nominal, and the final reading recorded. The sample is then subjected to additional increased loads until the data provides enough information to reasonably predict the potential for consolidation and/or swell under varying anticipated conditions.

- **HYDROMETER ANALYSIS** - The particle size distribution of the fine portion of a soil sample is determined in accordance with ASTM D1140. This test is run when the specific determination of the particle size distribution of the fine grained soils (-200) beyond standard classification (using -200 testing, in conjunction with Atterberg Limits) is required.

- **R VALUE** - This test is becoming more widely used for the design of pavement sections. Resistance ("R" Value) testing is performed by the California Materials Method No. 301 for base, sub-base, and base course. Three samples are prepared. Exudation pressures and "R" values are determined for each specimen. The graphically determined "R" value, at an exudation pressure of 300 psi, is reported.

- **SOLUBLE SULFATES** - The soluble sulfate content of representative samples is evaluated by standard geochemical techniques. California Materials Method No. 417.

- **SAND EQUIVALENT (S.E.)** - Sand Equivalent (S.E.) testing is performed in general accordance with AASHTO T176.

APPENDIX D

GRADING SPECIFICATIONS

Suggested Specifications For Placement of
Compacted Earth Fill and Backfill

TABLE OF CONTENTS

ATTACHMENTS

GRADING SPECIFICATIONS

GENERAL

A representative of the soils engineer should be on-site as the owner's representative to observe the placement of all compacted fill and backfill on the project. The soils engineer shall inspect all earth materials prior to their use, in addition to the methods of placement, and the degree of compaction obtained.

MATERIALS

Materials used for compacted fill and backfill shall be approved by the soils engineer prior to their use. Fill material, including rock, shall have a maximum dimension no greater than six inches (6"). Rock within fill should be dispersed to avoid nesting of rocks and creation of voids. In no case shall organic or other unsuitable material be used as fill or backfill material.

PREPARATION OF SUBGRADE

All topsoil, vegetation (including trees, major root systems, and brush), lumber, debris, rubbish, and other unsuitable material in areas to receive fill shall be removed to a depth satisfactory to the soils engineer and disposed of off-site. Removals shall extend a minimum of five feet (5') beyond the building footprint of all proposed structures. The surface of the area to be filled shall be scarified to a minimum depth of eight inches (8"), moistened or dried as necessary, and adequately compacted in a manner specified below. On slopes, a "keyway" must be excavated in accordance with Figure D-4 "Key and Benching Details", and approved before any fills are placed.

PLACING FILL

No fill shall be placed during weather conditions that would be adverse to the fill placement. All soil clods shall be reduced to six inches (6") or smaller size. Distribution of material in the fill shall be such as to preclude the formation of layers of material differing from the surrounding material. Each layer of fill shall be thoroughly mixed during placement to insure uniformity of material and moisture in each layer. Each layer shall have a maximum loose thickness of six to ten inches (6"-10"), and its surface shall be approximately horizontal. Each successive lift of fill placed on slopes should be benched into the slope, providing good bond between the fill and slope (see Figure D-3 "Side Hill Stability Fill Detail").

MOISTURE CONTROL

During compaction, the fill material in each layer shall be conditioned to a moisture content near or slightly above optimum, with the moisture content uniform throughout the fill. If, in the opinion of the soils engineer, the material placed as fill is too wet or dry to permit adequate compaction, it shall be removed and adequately dried or moisture conditioned prior to replacement and compaction.

COMPACTION

When an acceptable, uniform moisture content is obtained, each fill layer shall be compacted to applicable standards by a method acceptable to the soils engineer and as specified in the foregoing report. Compaction shall be performed by multiple passes with approved equipment suited to the soils being compacted. If a “sheep’s foot” roller is used, it shall be provided with cleaner bars attached in a manner that will prevent the accumulation of material between the tamper feet. The tamper feet should be able to provide an increase in effective weight.

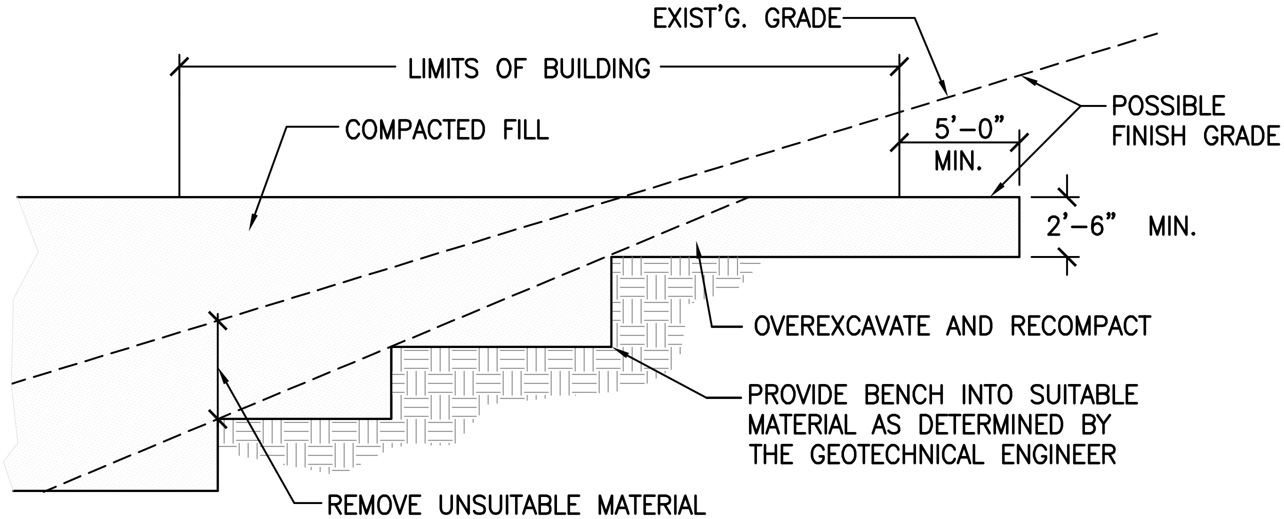
MOISTURE-DENSITY DETERMINATION

Representative samples of fill materials to be placed shall be furnished to the soils engineer by the contractor for determination of maximum density and optimum moisture content for these materials. Tests for this determination will be made using methods conforming to requirements of ASTM D698 or ASTM D1557. The results of these tests shall be the basis of control for all compaction effort.

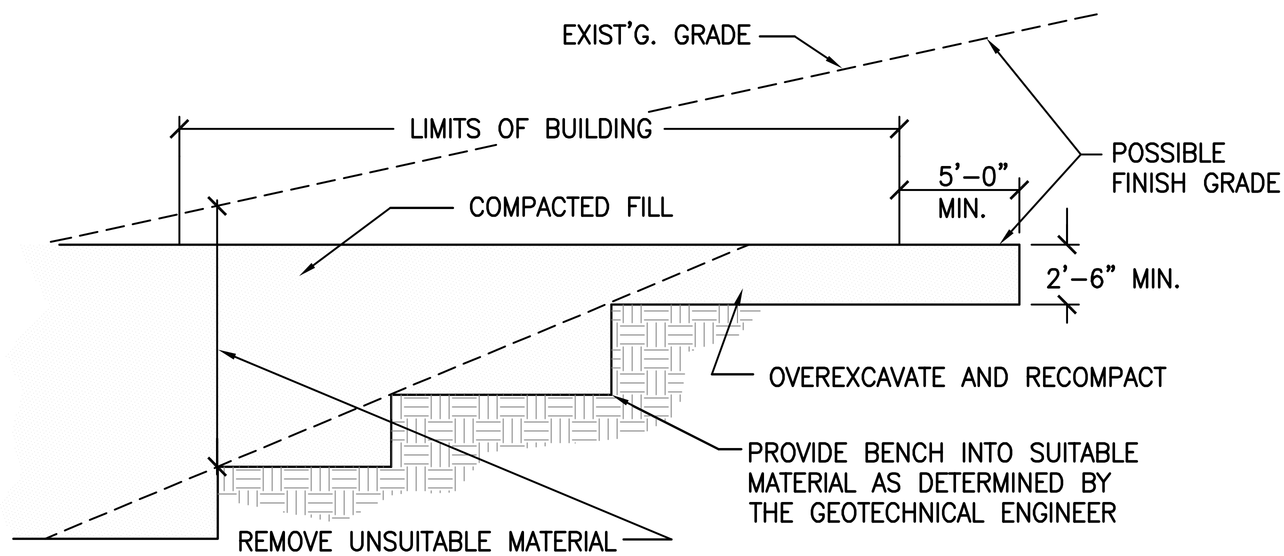
DENSITY TESTS

The density and moisture content of each layer of compacted fill will be determined by the soils engineer in accordance with ASTM D1556 or ASTM D2922. Any material found not to comply with the minimum specified density shall be recompacted until the required density is obtained. Sufficient density tests shall be made and the results submitted to support the soils engineer’s recommendations. The results of density tests will also be furnished to the owner, the project engineer, and the contractor by the soils engineer.

CUT-FILL LOT



CUT LOT



NOTE:

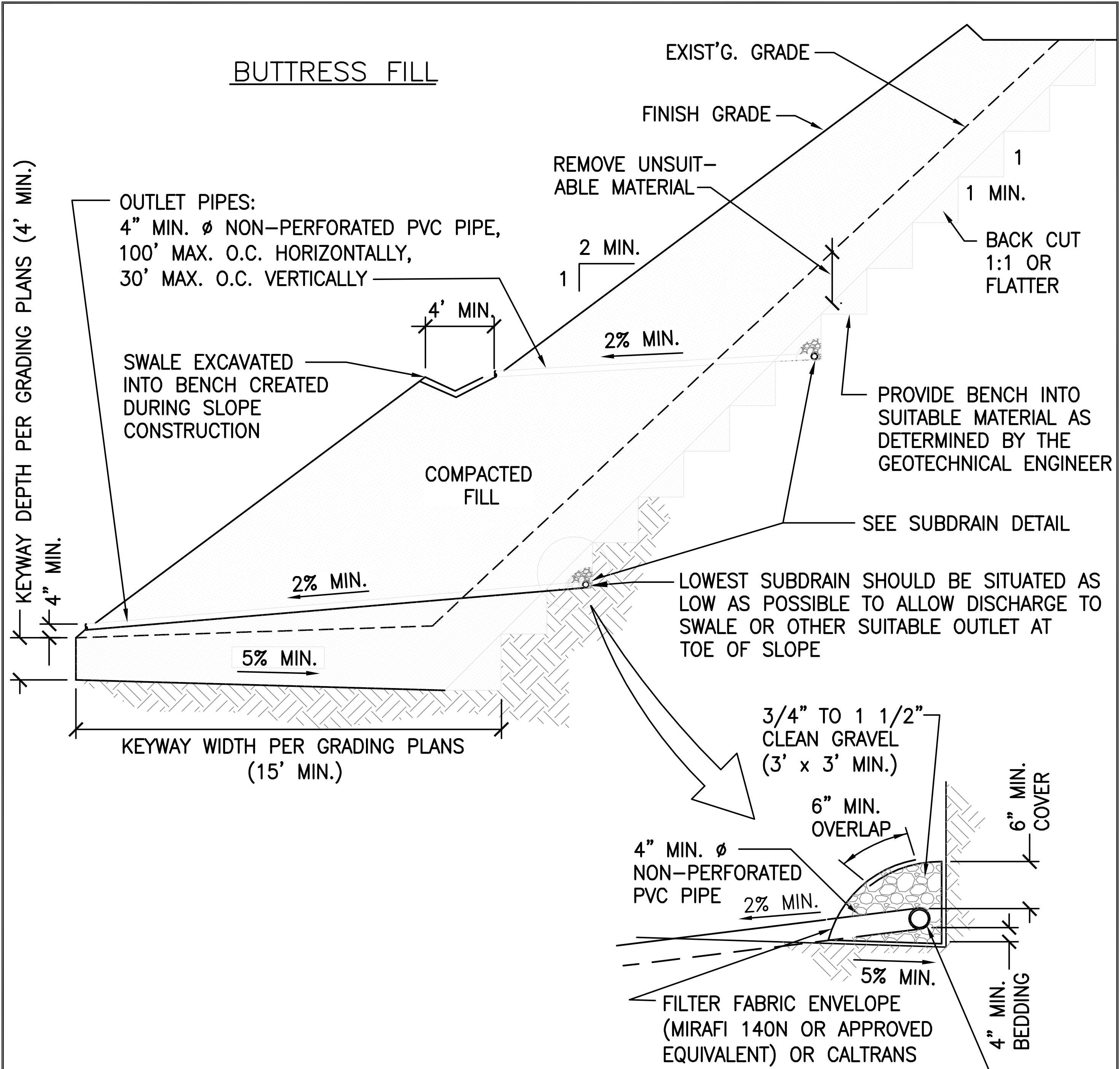
REMOVALS, KEYWAY DEPTH/WIDTH, AND SUBDRAIN RECOMMENDATIONS MAY BE MODIFIED BY THE GEOTECHNICAL ENGINEER BASED ON FIELD CONDITIONS.

TRANSITION LOT GRADING

ACCUTECH ENGINEERING

6253-6265-6275 MONTEZUMA ROAD
SAN DIEGO, CA 92115

DATE:	FIGURE NO.:	PROJECT NO.:
08/30/2018	D- 1	18423-1



NOTE:

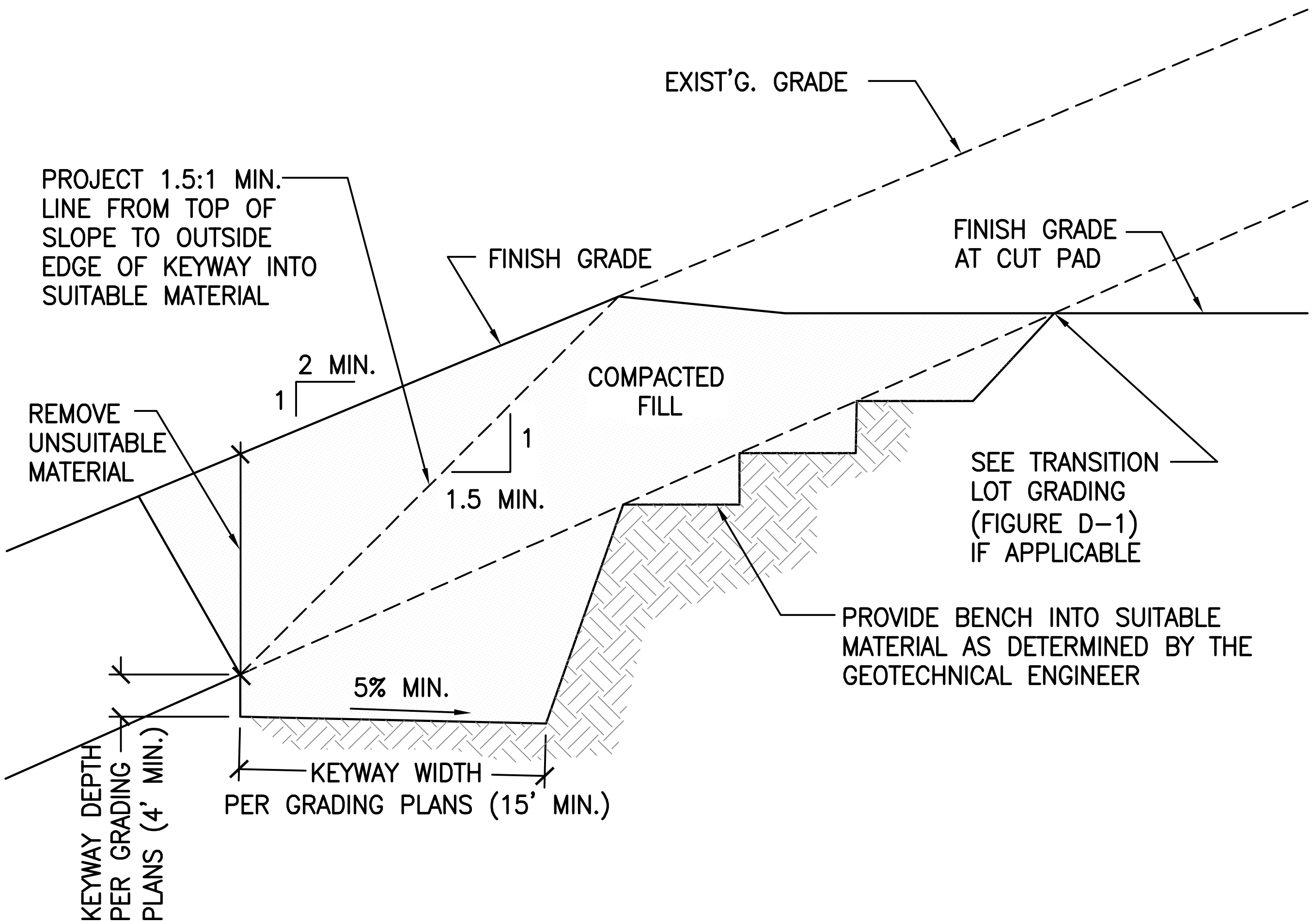
REMOVALS, KEYWAY DEPTH/WIDTH, AND SUBDRAIN RECOMMENDATIONS MAY BE MODIFIED BY THE GEOTECHNICAL ENGINEER BASED ON FIELD CONDITIONS.

BUTTRESS FILL & SUBDRAIN TRENCH

ACCUTECH ENGINEERING

6253-6265-6275 MONTEZUMA ROAD
SAN DIEGO, CA 92115

DATE:	FIGURE NO.:	PROJECT NO.:
08/30/2018	D- 2	18423-1



NOTE:

REMOVALS, KEYWAY DEPTH/WIDTH, AND SUBDRAIN RECOMMENDATIONS MAY BE MODIFIED BY THE GEOTECHNICAL ENGINEER BASED ON (EXPOSED) FIELD CONDITIONS.

NOT TO SCALE

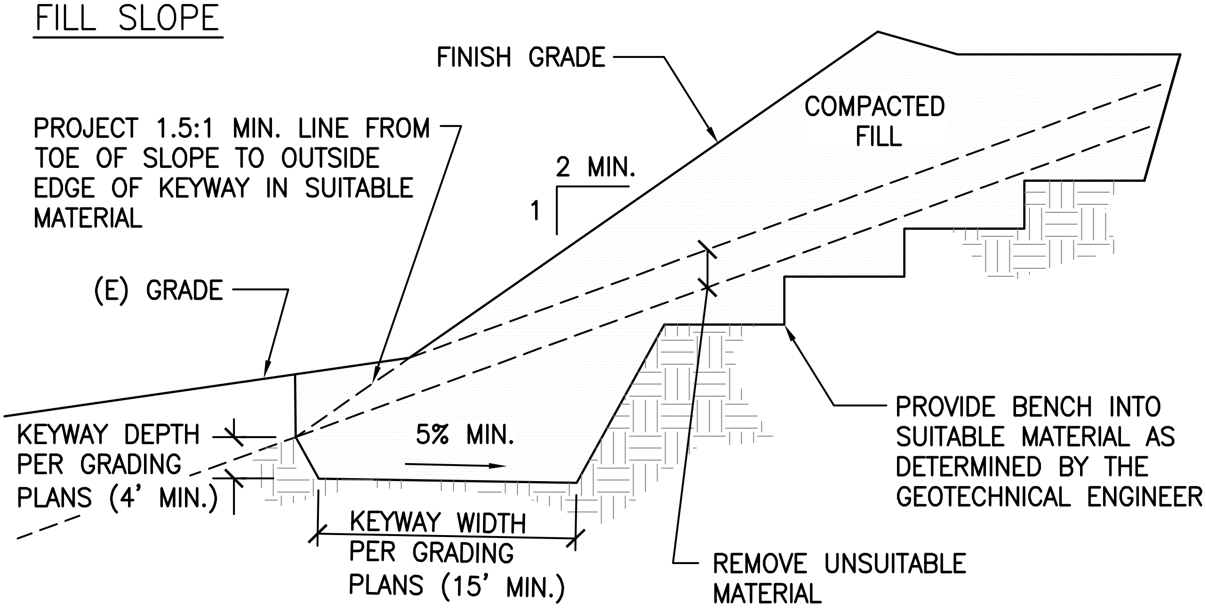
STABILITY FILL

ACCUTECH ENGINEERING

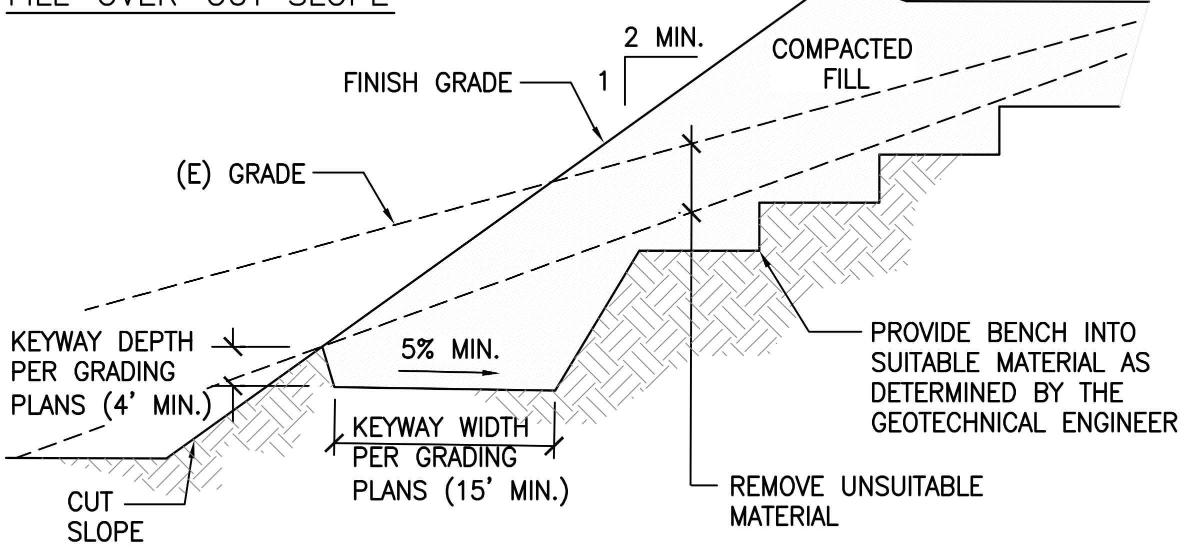
6253-6265-6275 MONTEZUMA ROAD
SAN DIEGO, CA 92115

DATE:	FIGURE NO.:	PROJECT NO.:
08/30/2018	D- 3	18423-1

FILL SLOPE



FILL-OVER-CUT SLOPE



NOTE:

REMOVALS, KEYWAY DEPTH/WIDTH, AND SUBDRAIN RECOMMENDATIONS MAY BE MODIFIED BY THE GEOTECHNICAL ENGINEER BASED ON FIELD CONDITIONS.

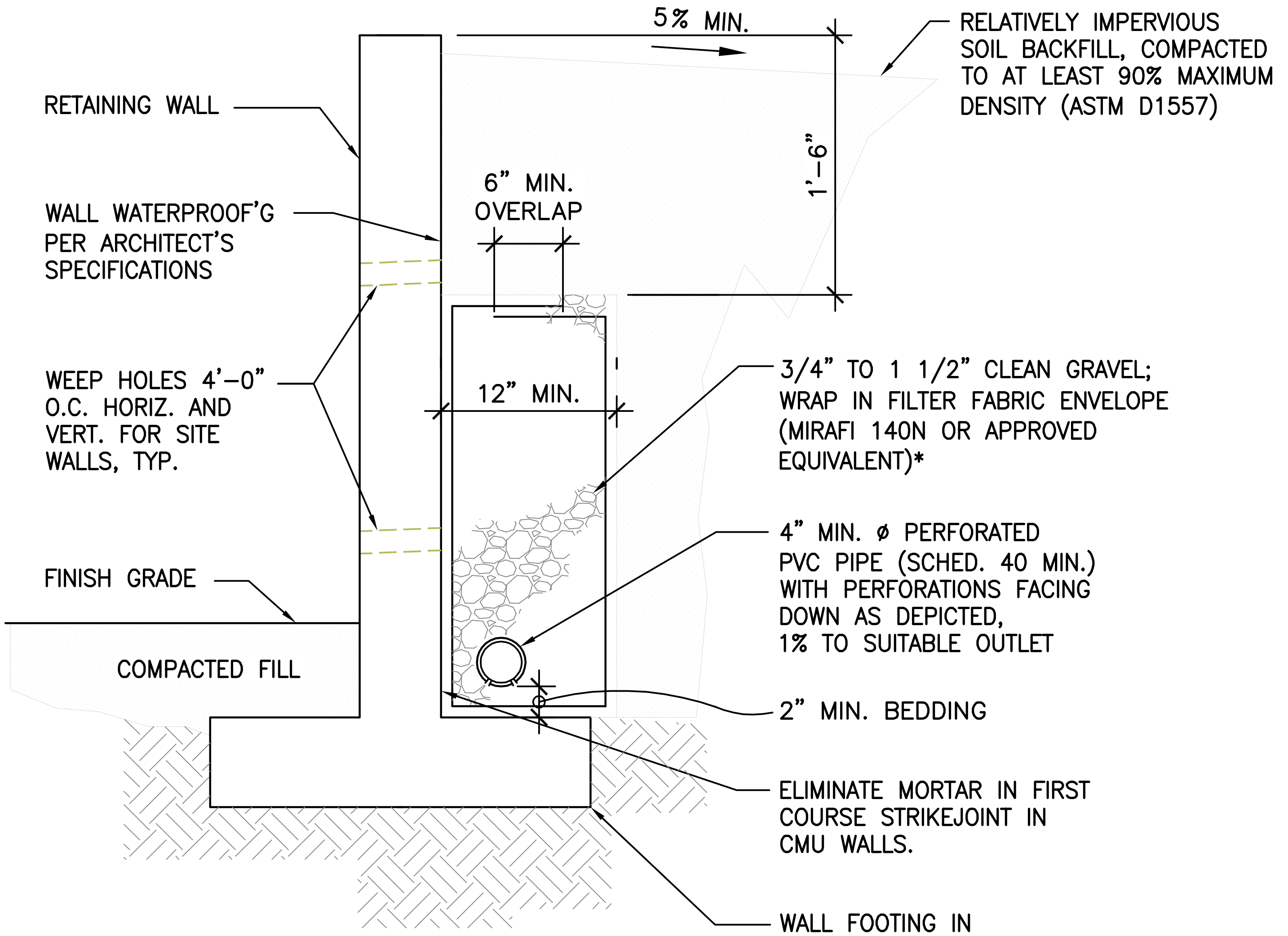
NOT TO SCALE

FILL SLOPES

ACCUTECH ENGINEERING

6253-6265-6275 MONTEZUMA ROAD
SAN DIEGO, CA 92115

DATE:	FIGURE NO.:	PROJECT NO.:
08/30/2018	D-4	18423-1



SPECIFICATIONS FOR CALTRANS
CLASS 2 PERMEABLE MATERIAL

U.S. STANDARD SIEVE SIZE	% PASSING
1"	100
3/4"	90–100
3/8"	40–100
No. 4	25–40
No. 8	18–33
No. 30	5–15
No. 50	0–7
No. 200	0–3

SAND EQUIVALENT >75

NOTE:
IF CALTRANS CLASS 2 PERMEABLE MATERIAL (SEE GRADATION TO LEFT) IS USED IN PLACE OF 3/4" TO 1 1/2" GRAVEL, FILTER FABRIC IS NOT REQUIRED. CALTRANS CLASS 2 PERMEABLE MATERIAL MUST BE COMPACTED TO AT LEAST 90% MAXIMUM DENSITY (ASTM D1557)

NOT TO SCALE

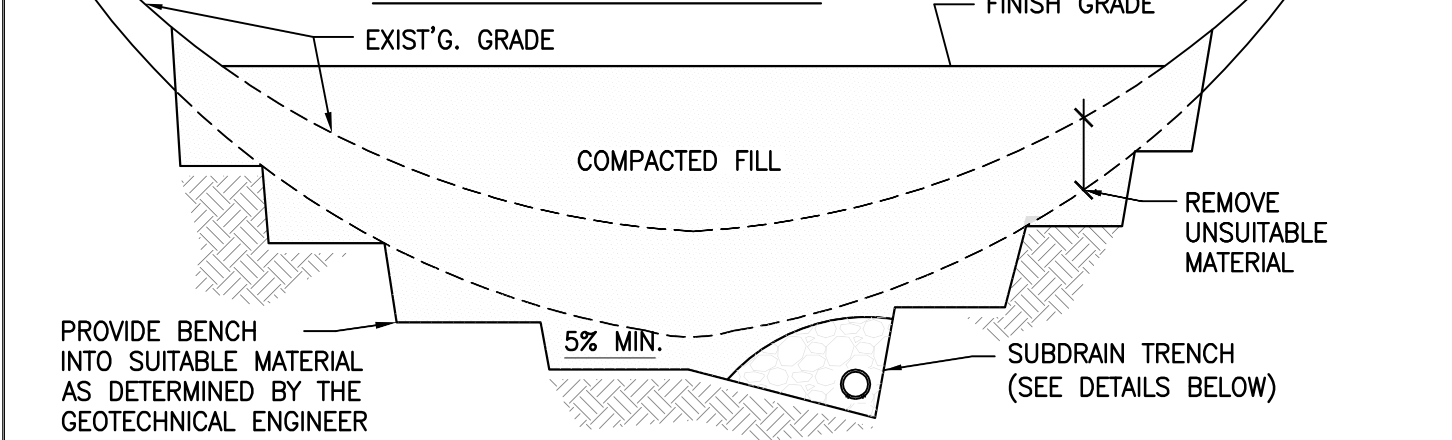
SITE RETAINING WALL DRAINAGE

ACCUTECH ENGINEERING

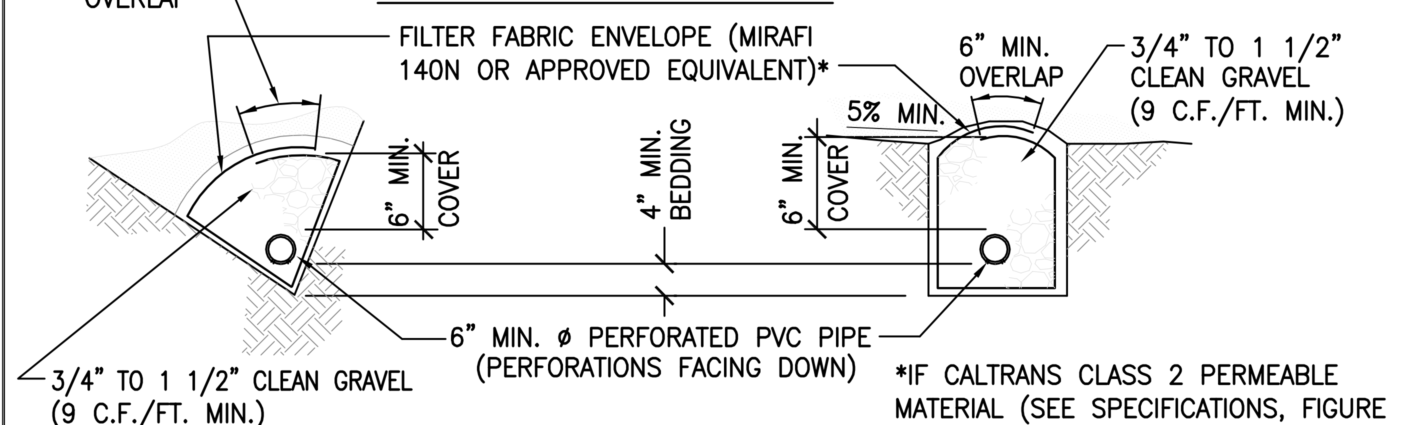
6253-6265-6275 MONTEZUMA ROAD
SAN DIEGO, CA 92115

DATE:	FIGURE NO.:	PROJECT NO.:
08/30/2018	D- 5	18423-1

CANYON SUBDRAIN SECTION

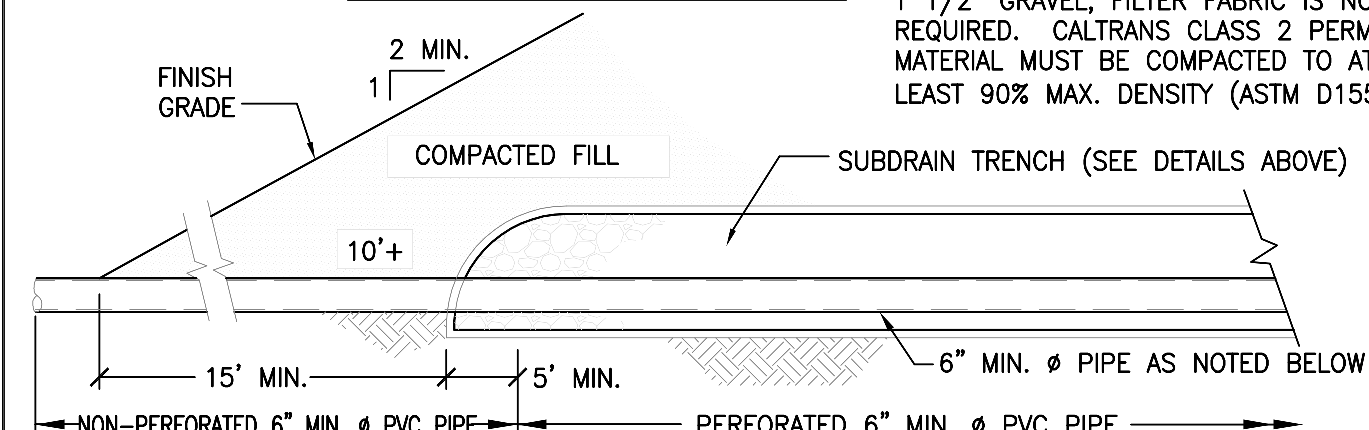


SUBDRAIN TRENCH DETAILS



*IF CALTRANS CLASS 2 PERMEABLE MATERIAL (SEE SPECIFICATIONS, FIGURE D-6) IS USED IN PLACE OF 3/4" TO 1 1/2" GRAVEL, FILTER FABRIC IS NOT REQUIRED. CALTRANS CLASS 2 PERMEABLE MATERIAL MUST BE COMPACTED TO AT LEAST 90% MAX. DENSITY (ASTM D1557).

CANYON SUBDRAIN ELEVATION



NOTE:

REMOVALS, KEYWAY DEPTH/WIDTH, & SUBDRAIN RECOMMENDATIONS MAY BE MODIFIED BY THE GEOTECHNICAL ENGINEER BASED ON FIELD CONDITIONS.

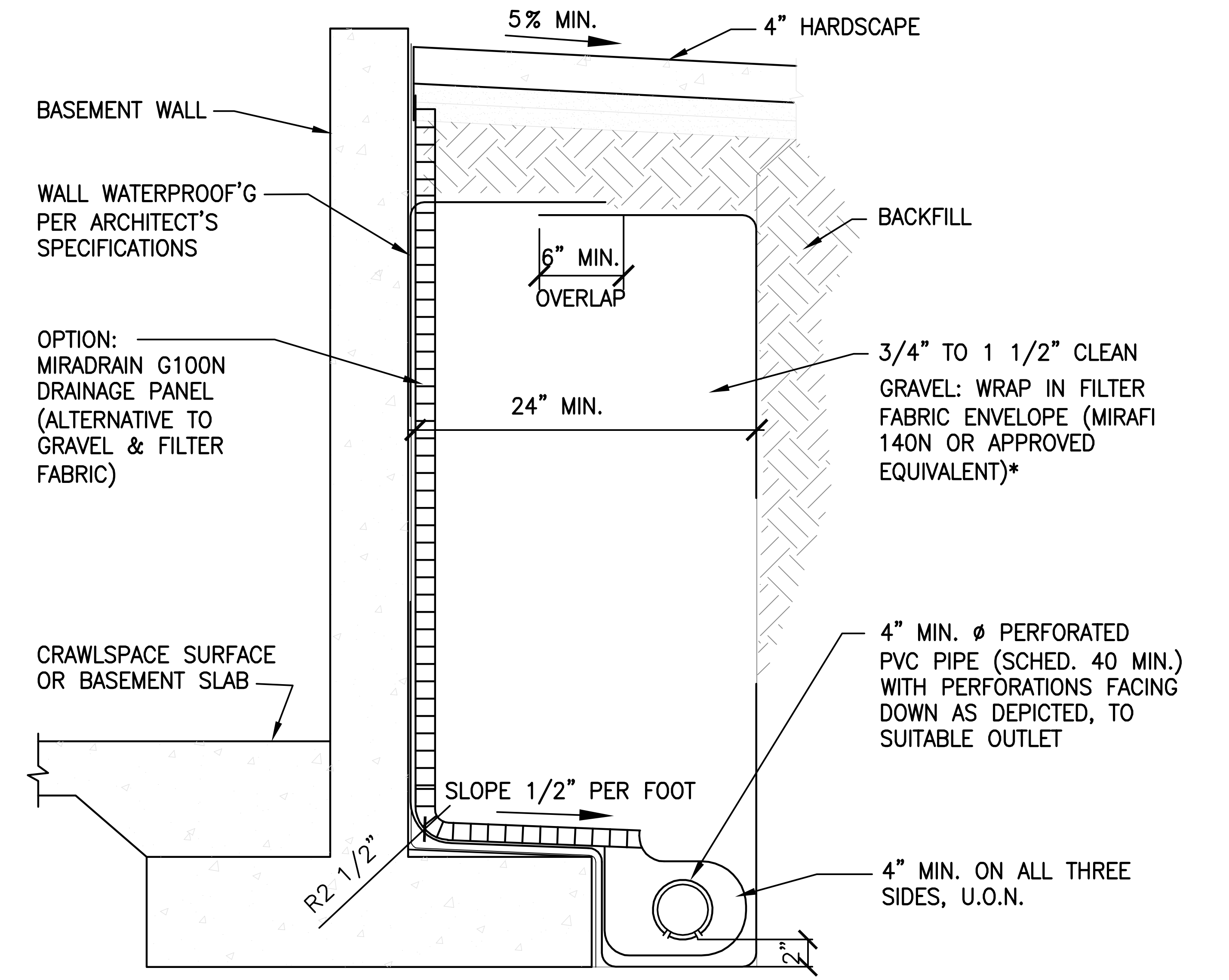
NOT TO SCALE

CANYON SUBDRAIN

ACCUTECH ENGINEERING

6253-6265-6275 MONTEZUMA ROAD
SAN DIEGO, CA 92115

DATE:	FIGURE NO.:	PROJECT NO.:
08/30/2018	D-6	18423-1



SPECIFICATIONS FOR CALTRANS CLASS 2 PERMEABLE MATERIAL

U.S. STD. SIEVE SIZE	% PASSING
1"	100
3/4"	90–100
3/8"	40–100
No. 4	25–40
No. 8	18–33
No. 30	5–15
No. 50	0–7
No. 200	0–3

SAND EQUIVALENT >75

* IF CALTRANS CLASS 2 PERMEABLE MATERIAL (SEE GRADATION TO LEFT) IS USED IN PLACE OF 3/4" TO 1 1/2" GRAVEL, FILTER FABRIC IS NOT REQUIRED. CALTRANS CLASS 2 PERMEABLE MATERIAL MUST BE COMPACTED TO AT LEAST 90% MAXIMUM DENSITY (ASTM D1557)

NOT TO SCALE

STEMWALL SUBDRAIN/BASEMENT WALL DRAINAGE

ACCUTECH ENGINEERING

6253-6265-6275 MONTEZUMA ROAD
SAN DIEGO, CA 92115

DATE:	FIGURE NO.:	PROJECT NO.:
08/30/2018	D- 7	18423-1

APPENDIX E

GEOLOGIC RECONNAISSANCE

Prepared by Michael W. Hart
Certified Engineering Geologist
CEG #706

`MICHAEL W. HART
ENGINEERING GEOLOGIST
P.O. BOX 261227 • SAN DIEGO • CALIFORNIA • 92196 • 858 578-4672

August 6, 2018
File No. 1078-2018

Accutech Engineering Systems
3435 Carleton Street
San Diego, California
92106

Attn: Robert Randall

Subject: 6253, 6265, & 6275 Montezuma Road
San Diego, California
GEOLOGIC RECONNAISSANCE

Dear Mr. Randall:
In accordance with our agreement I have completed a geologic reconnaissance of the residential property located at 6253, 6265, and 6275 Montezuma Road, San Diego, California. The results of this study indicate that the property is underlain by dense, well-cemented marine terrace deposits. This unit is locally overlain by clayey topsoils.

A review of aerial photographs and geologic literature indicates that the property is not underlain by an active or potentially active fault. In addition, the results of this study indicate that the property is not underlain by an ancient landslide.

If you have any questions concerning the findings or conclusions of the report please contact me at your convenience.

Very truly yours,

REGISTERED GEOLOGIST
MICHAEL W. HART
NO. CEG706
CERTIFIED
ENGINEERING
GEOLOGIST
EXP. 10/31/18
STATE OF CALIFORNIA

Michael W. Hart
CEG 706

GEOLOGIC RECONNAISSANCE
6253, 6265, and 6275 MONTEZUMA ROAD
San Diego, California

INTRODUCTION

This report presents the results of a geologic reconnaissance for the proposed multi-story residential structure located at 6253, 6265, and 6275 Montezuma Road in San Diego, California (Figure 1). This report is a reconnaissance level study whose purpose is to describe the geologic characteristics of the site as well as the potential geologic hazards to which the site may be susceptible. The scope of work included geologic mapping of the site, inspection of two exploratory trenches placed by Accutech Engineering Systems during the geotechnical investigation, and review of published geologic literature.

FIELD WORK

Fieldwork performed for this study consisted of geologic observation of exploratory test trench exposures by Accutech Engineering during the geotechnical investigation and mapping of natural outcrops on and near the site utilizing the project site plan.

SITE DESCRIPTION AND PROPOSED PROJECT

The site is located on the southwest corner of Montezuma Road and 63rd Street in the College area of the City of San Diego. Topographically, the site consists of a nearly level graded building pad constructed at an approximate elevation of 465 feet. Drainage on the site is by sheet flow toward Montezuma Road and 63rd Street. Project plans are not yet complete, however, it is understood that preliminary plans call for demolition of the three existing single structures that currently exist on the property and construction of a multi-story residential building with partial basement parking.

GENERAL GEOLOGY AND GEOLOGIC SETTING

The project is situated in the coastal section of the Peninsular Ranges Geomorphic Province. The coastal section is underlain by a thick sequence of primarily marine clastic sediments eroded from the Peninsular Ranges as a result of tectonic uplift beginning in the Cretaceous Period approximately 60 million years ago. That portion of the coastal province in which the site is located is underlain by Tertiary-aged marine sediments consisting of the essentially flat lying

Marine Terrace Deposits (Lindavista Formation, Kennedy, 1975, Very old Paralic Deposits, Kennedy and Tan, 2008).

During Quaternary time approximately 1.5 million years before present, regional uplift and faulting near the present day coastline began that continues to the present. The La Nacion fault zone, approximately one mile west of the site, is the closest significant fault and is structurally related to the active Rose Canyon fault zone further to the west. Together, these faults define a wide and complexly faulted basin occupied by San Diego Bay and a narrow section of the continental shelf west of the Silver Strand. The site's location with respect to the La Nacion and other local faults is discussed in the Geologic Hazards section of this report.

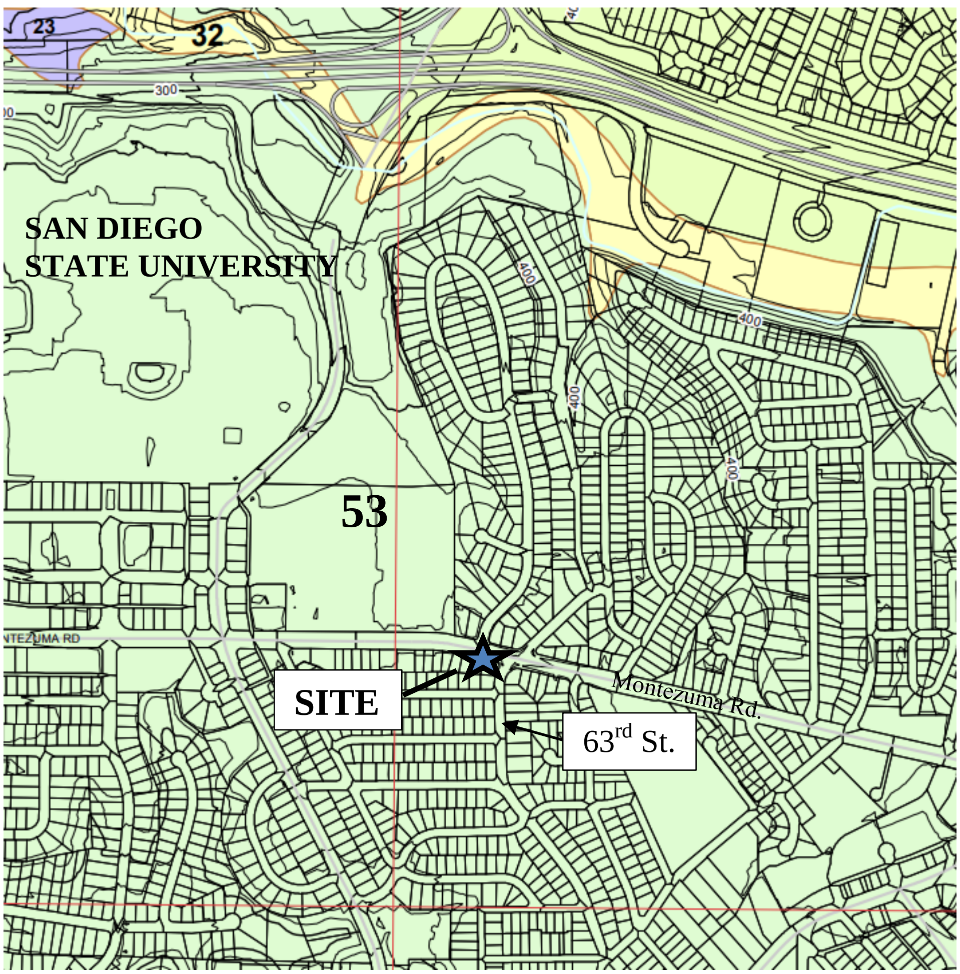


Figure 1. Site location and Geologic Hazard Category map (after City of San Diego Geologic Hazards and Fault Maps).
53: Level to sloping terrain, unfavorable geologic structure, low to moderate risk.

STRATIGRAPHY

Geologic units underlying the site are mapped by Kennedy (1975) as the Lindavista Formation. Later maps depict this unit as Very Old Paralic Deposits (Kennedy and Tan, 2008). These formational materials are locally overlain by surficial deposits consisting locally of a thin veneer of fill and topsoil described in older to younger order in the following paragraphs.

Lindavista Formation (Qln)

The early Pleistocene-aged Lindavista Formation (Very Old Paralic Deposits) is approximately 20 feet thick in the college area and overlies the Stadium Conglomerate. The Lindavista Formation in this area as evidenced by exploratory test pits excavated during the geotechnical investigation consists of moderately to very well-cemented orange-brown clayey sand and cobble conglomerate. Topsoils were absent in Trenches 1 and 2 probably as a result of grading during the original development of the properties. In Trench 3 near 63rd Street a topsoil remnant approximately 1.5 feet thick is present that is composed of stiff, brown, sandy clay. Trenches 1 and 2 were terminated at depths of approximately 5 feet due to the presence of cemented sandstone or conglomerate. Trench 3 was easily excavated to a depth of six feet.

GEOLOGIC STRUCTURE:

The geologic units underlying the site as evidenced by regional geologic mapping are essentially horizontally stratified. Inspection of limited outcrops as well as a study of aerial photographs indicates no evidence of on-site faulting or lineaments suggestive of possible faulting.

GEOLOGIC HAZARDS

Potential geologic hazards considered in this report include the potential for surface faulting, liquefaction, seismically induced settlement, landsliding, and seismic shaking.

Local Faulting:

According to mapping by Kennedy and Tan (1975) and the Seismic Hazard Maps of the City of San Diego, the site is not located on or adjacent to a known fault. The closest mapped potentially active fault is the La Nacion fault approximately 1.1 miles to the west. This fault extends north from the United States/Mexico border to the vicinity of Montezuma Road near San Diego State University. The La Nacion fault displaces Quaternary stream terrace materials dated at less than 125,000 years before present as well as a 125,000 yr. old near-shore marine sandstone that may be correlative with the Bay Point Formation. Although there are many strands making up the fault zone, the lack of geomorphic expression throughout its length suggests that none of the faults making up this wide fault zone has been active during the Holocene.

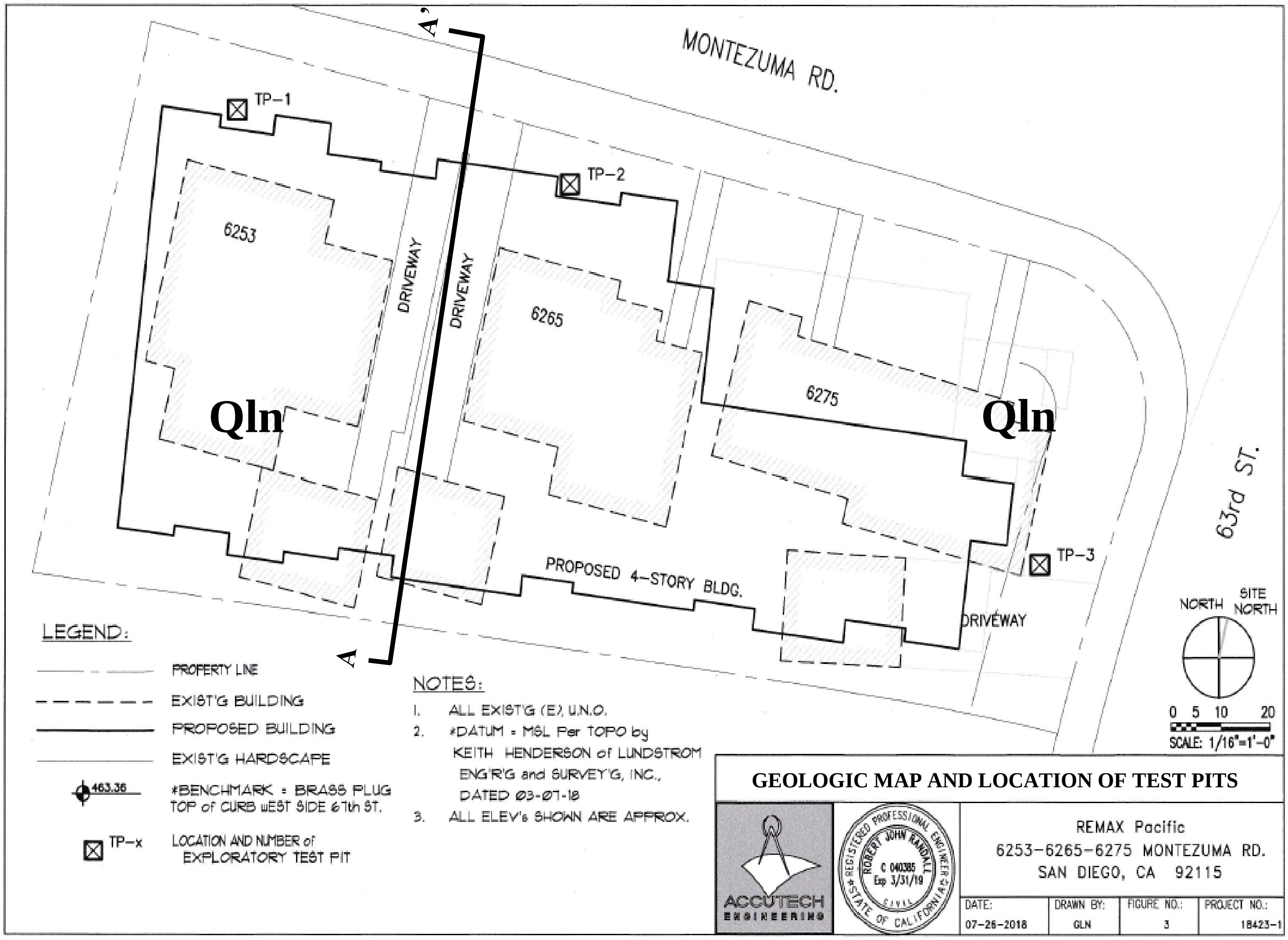


Figure 2. Geologic Map and location of Test Pits by Accutech Engineering Systems
Qln: Lindavista Formation (very old Paralic Deposits)

Regional Faulting and Seismicity:

The site will be affected by seismic shaking as a result of earthquakes on major active faults located throughout the southern California area. The nearest of these fault systems, the Rose Canyon fault (Mission Bay Segment) lies approximately seven miles to the west and is the most significant fault to the site with respect to the potential for seismic activity. The Del Mar segment extends from La Jolla to the vicinity of Oceanside. Recent paleoseismic trenching on the Rose Canyon fault in the Old Town area of San Diego by Singleton, Rockwell, and others (2017) indicates that the Rose Canyon fault had sustained activity throughout the Holocene and into the Historical period. Their study indicated evidence of four surface rupturing seismic events in the last 3500 years with the most recent event occurring in 1862. As a result of their study they calculated an average recurrence interval for surface rupturing events of 675 +/- 428 yrs. Such an event could produce peak ground accelerations at the site of approximately 0.40g (Joyner and Boore, 1982). Other active faults, the Elsinore, San Jacinto, and San Andreas faults lie approximately 35, 60, and 85 miles, respectively, to the east.

Liquefaction and Seismically Induced Settlement:

Formational soils underlying the undocumented fills in the proposed building site consist of dense Pleistocene-aged cobble conglomerate and cemented sandstone. These soils, as well as fill soils, if properly compacted, are not considered susceptible to seismically induced liquefaction or excessive settlement.

Landsliding and Slope Stability:

A review of topographic maps and geologic literature (Kennedy, 1975) indicates there is no geomorphic or geologic evidence to suggest the presence of ancient deep-seated landsliding on or adjacent to the site. The Landslide Hazards map for the La Mesa Quadrangle by Tan (1995) indicates the site lies within Subarea 3-1. Subarea 3-1 is defined as containing slopes that are at or near their stability limits due a combination of weak materials and steep slopes. Such areas typically do not currently contain landslide deposits but can be expected to fail locally when adversely modified.

CONCLUSIONS AND RECOMMENDATIONS

1. The proposed building site is underlain by well-cemented sand and cobble of the Lindavista Formation (very old paralic deposits). Current development plans call for partial basement parking that will be at the approximate level of Montezuma Road (+/- 5 ft. below existing grade). At this elevation the foundations should lie below any existing fill.

2. The closest significant fault is a branch of the La Nacion fault located approximately 1.1 miles west of the site. The closest active fault to the property is the Rose Canyon fault that lies approximately seven miles to the west.

3. A study of aerial photographs and topographic maps indicates there are no deep-seated landslides on or adjacent to the property. In addition, it is concluded that the site is not located on or adjacent to an active or potentially active fault.

REFERENCES

Anderson, J. G., Rockwell, T., and Agnew, D.C., 1989, A study of the seismic hazard in San Diego, Earthquake Spectra, vol. 5(2), pp 229-333.

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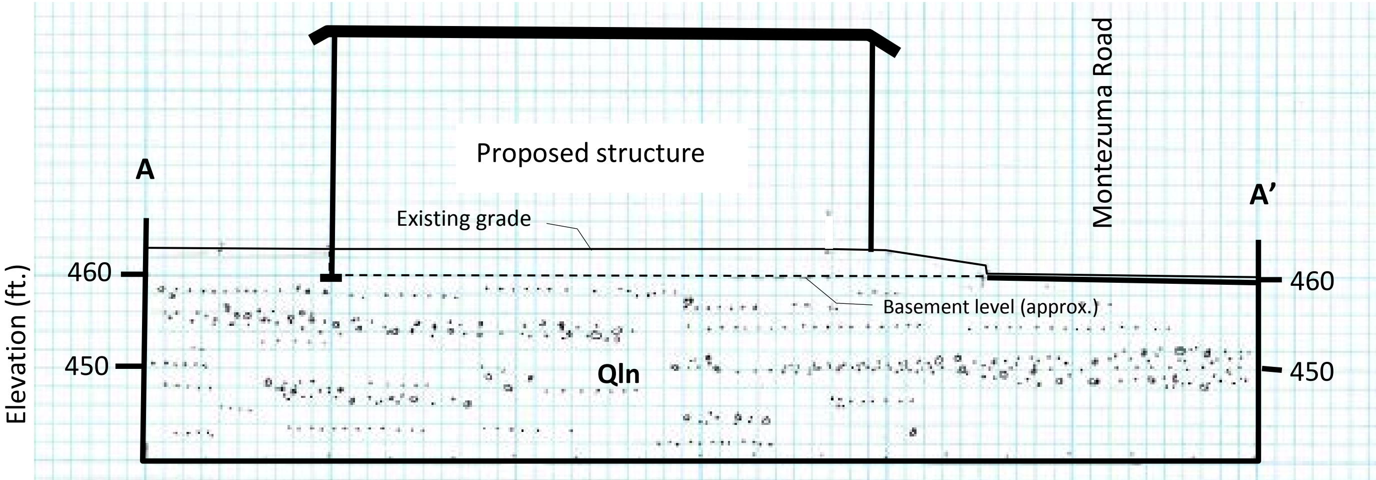
Kennedy, M.P. and Tan, S.S., 1975, Character and recency of faulting, San Diego metropolitan area, California, California Div. Mines and Geol. Special Rept. 123, pp. 33.

Kennedy, M. P., and Tan, S.S., 2008, Geology of the San Diego 30 X 60 minute Quadrangle, San Diego, California., California Geologic Survey Regional Geologic Map Series, 1:100,000 Scale; Map No. 3, Sheet 1.

Lindvall, S.C., and Rockwell, T.K., 1995, Holocene activity of the Rose Canyon fault zone in San Diego, California, Jour. Geophysical Research, vol. 100, no. B12, Pages 24,121-24-132.

Tan, S.S., 1995, Landslide Hazards in the southern part of the San Diego metropolitan area, San Diego County, California: Landslide Hazard Identification Map No. 33.



GEOLOGIC SECTION A – A’
6353, 6265, 6275 MONTEZUMA ROAD, SAN DIEGO, CALIFORNIA

LEGEND

Qln: Lindavista Formation (Very Old Paralic Deposits)