CLIMATE ACTION PLAN CONSISTENCY CHECKLIST INTRODUCTION

In December 2015, the City adopted a Climate Action Plan (CAP) that outlines the actions that City will undertake to achieve its proportional share of State greenhouse gas (GHG) emission reductions. The purpose of the Climate Action Plan Consistency Checklist (Checklist) is to, in conjunction with the CAP, provide a streamlined review process for proposed new development projects that are subject to discretionary review and trigger environmental review pursuant to the California Environmental Quality Act (CEQA).[1]

Analysis of GHG emissions and potential climate change impacts from new development is required under CEQA. The CAP is a plan for the reduction of GHG emissions in accordance with CEQA Guidelines Section 15183.5. Pursuant to CEQA Guidelines Sections 15064(h)(3), 15130(d), and 15183(b), a project's incremental contribution to a cumulative GHG emissions effect may be determined not to be cumulatively considerable if it complies with the requirements of the CAP.

This Checklist is part of the CAP and contains measures that are required to be implemented on a project-by-project basis to ensure that the specified emissions targets identified in the CAP are achieved. Implementation of these measures would ensure that new development is consistent with the CAP's assumptions for relevant CAP strategies toward achieving the identified GHG reduction targets. Projects that are consistent with the CAP as determined through the use of this Checklist may rely on the CAP for the cumulative impacts analysis of GHG emissions. Projects that are not consistent with the CAP must prepare a comprehensive project-specific analysis of GHG emissions, including quantification of existing and projected GHG emissions and incorporation of the measures in this Checklist to the extent feasible. Cumulative GHG impacts would be significant for any project that is not consistent with the CAP.

The Checklist may be updated to incorporate new GHG reduction techniques or to comply with later amendments to the CAP or local, State, or federal law.

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[1] Certain projects seeking ministerial approval may be required to complete the Checklist. For example, projects in a Community Plan Implementation Overlay Zone may be required to use the Checklist to qualify for ministerial level review. See Supplemental Development Regulations in the project's community plan to determine applicability.

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CAP CONSISTENCY CHECKLIST SUBMITTAL APPLICATION

- The Checklist is required only for projects subject to CEQA review.[2]
- If required, the Checklist must be included in the project submittal package. Application submittal procedures can be found in Chapter 11: Land Development Procedures of the City's Municipal Code.
- The requirements in the Checklist will be included in the project's conditions of approval.
- The applicant must provide an explanation of how the proposed project will implement the requirements described herein to the satisfaction of the Planning Department.

Application Information

Contact Information

Project No./Name: Hershfield Trust / 603740

Property Address: 8230 Prestwick Drive, La Jolla CA 92037

Applicant Name/Co.: Claude-Anthony Marengo / Marengo Morton Architects

Contact Phone: 619-417-1111 Contact Email: cmarengo@san.rr.com

Was a consultant retained to complete this checklist? ☐ Yes ☐ No If Yes, complete the following

Consultant Name: Contact Phone:

Company Name: Contact Email:

Project Information

1. What is the size of the project (acres)? 0.47 Acres
2. Identify all applicable proposed land uses:
   - ■ Residential (indicate # of single-family units): 1
   - ☐ Residential (indicate # of multi-family units):
   - ☐ Commercial (total square footage):
   - ☐ Industrial (total square footage):
   - ☐ Other (describe):
3. Is the project or a portion of the project located in a Transit Priority Area? ☐ Yes ■ No
4. Provide a brief description of the project proposed:

To demolish an existing one-story single-family residence constructed in 1985 and construct a one-story single-family residence with basement, decks, and back-yard swimming pool totaling 12,909 square-feet (5,537 square-feet consists of basement). The 0.4480 acres (19,550 sq. ft.) project site is located at 8230 Prestwick Drive.

[2] Certain projects seeking ministerial approval may be required to complete the Checklist. For example, projects in a Community Plan Implementation Overlay Zone may be required to use the Checklist to qualify for ministerial level review. See Supplemental Development Regulations in the project's community plan to determine applicability.

CAP CONSISTENCY CHECKLIST QUESTIONS

Step 1: Land Use Consistency

The first step in determining CAP consistency for discretionary development projects is to assess the project's consistency with the growth projections used in the development of the CAP. This section allows the City to determine a project's consistency with the land use assumptions used in the CAP.

Step 1: Land Use Consistency		
Checklist Item (Check the appropriate box and provide explanation and supporting documentation for your answer)	Yes	No
A. Is the proposed project consistent with the existing General Plan and Community Plan land use and zoning designations?;[3] OR, B. If the proposed project is not consistent with the existing land use plan and zoning designations, and includes a land use plan and/or zoning designation amendment, would the proposed amendment result in an increased density within a Transit Priority Area (TPA)[4] and implement CAP Strategy 3 actions, as determined in Step 3 to the satisfaction of the Development Services Department?; OR, C. If the proposed project is not consistent with the existing land use plan and zoning designations, does the project include a land use plan and/or zoning designation amendment that would result in an equivalent or less GHG-intensive project when compared to the existing designations?	☑	☐

If "***Yes***," proceed to Step 2 of the Checklist. For question B above, complete Step 3. For question C above, provide estimated project emissions under both existing and proposed designation(s) for comparison. Compare the maximum buildout of the existing designation and the maximum buildout of the proposed designation.

If "***No***," in accordance with the City's Significance Determination Thresholds, the project's GHG impact is significant. The project must nonetheless incorporate each of the measures identified in Step 2 to mitigate cumulative GHG emissions impacts unless the decision maker finds that a measure is infeasible in accordance with CEQA Guidelines Section 15091. Proceed and complete Step 2 of the Checklist.

The project is consistent with the Single-family (SF) Zone of the La Jolla Shores Planned District (LJSPD), and the Coastal Overlay (Non-Appealable Area 2), Coastal Height Limitation Overlay, and the Parking Impact (Coastal Impact Area) Overlay Zones, within the La Jolla Community Plan and Local Coastal Program (LJCP) land use plan.

[3] This question may also be answered in the affirmative if the project is consistent with SANDAG Series 12 growth projections, which were used to determine the CAP projections, as determined by the Planning Department.

[4] This category applies to all projects that answered in the affirmative to question 3 on the previous page: Is the project or a portion of the project located in a transit priority area.

Step 2: CAP Strategies Consistency

The second step of the CAP consistency review is to review and evaluate a project's consistency with the applicable strategies and actions of the CAP. Step 2 only applies to development projects that involve permits that would require a certificate of occupancy from the Building Official or projects comprised of one and two family dwellings or townhouses as defined in the California Residential Code and their accessory structures.[5] All other development projects that would not require a certificate of occupancy from the Building Official shall implement Best Management Practices for construction activities as set forth in the Greenbook (for public projects).

Step 2: CAP Strategies Consistency			
Checklist Item (Check the appropriate box and provide explanation for your answer)	Yes	No	N/A
Strategy 1: Energy & Water Efficient Buildings			
1. *Cool/Green Roofs.* • Would the project include roofing materials with a minimum 3-year aged solar reflection and thermal emittance or solar reflection index equal to or greater than the values specified in the voluntary measures under California Green Building Standards Code (Attachment A)?; <u>OR</u> • Would the project roof construction have a thermal mass over the roof membrane, including areas of vegetated (green) roofs, weighing at least 25 pounds per square foot as specified in the voluntary measures under California Green Building Standards Code?; <u>OR</u> • Would the project include a combination of the above two options? Check "N/A" only if the project does not include a roof component. The project will provide roofing materials with a minimum 3-year aged solar reflection and thermal emittance or solar reflection index equal to or greater than the values specified in the voluntary measures under California Green Building Standards Code.	☑	☐	☐

[5] Actions that are not subject to Step 2 would include, for example: 1) discretionary map actions that do not propose specific development, 2) permits allowing wireless communication facilities, 3) special events permits, 4) use permits or other permits that do not result in the expansion or enlargement of a building (e.g., decks, garages, etc.), and 5) non-building infrastructure projects such as roads and pipelines. Because such actions would not result in new occupancy buildings from which GHG emissions reductions could be achieved, the items contained in Step 2 would not be applicable.

2. *Plumbing fixtures and fittings* With respect to plumbing fixtures or fittings provided as part of the project, would those low-flow fixtures/appliances be consistent with each of the following: Residential buildings: • Kitchen faucets: maximum flow rate not to exceed 1.5 gallons per minute at 60 psi; • Standard dishwashers: 4.25 gallons per cycle; • Compact dishwashers: 3.5 gallons per cycle; and • Clothes washers: water factor of 6 gallons per cubic feet of drum capacity? Nonresidential buildings: • Plumbing fixtures and fittings that do not exceed the maximum flow rate specified in Table A5.303.2.3.1 (voluntary measures) of the California Green Building Standards Code (See Attachment A); and • Appliances and fixtures for commercial applications that meet the provisions of Section A5.303.3 (voluntary measures) of the California Green Building Standards Code (See Attachment A)? Check "N/A" only if the project does not include any plumbing fixtures or fittings. The project will provide all low-flow plumbing fixtures/appliances.	☑	☐	☐

Strategy 3: Bicycling, Walking, Transit & Land Use			
3. *Electric Vehicle Charging* • Multiple-family projects of 17 dwelling units or less: Would 3% of the total parking spaces required, or a minimum of one space, whichever is greater, be provided with a listed cabinet, box or enclosure connected to a conduit linking the parking spaces with the electrical service, in a manner approved by the building and safety official, to allow for the future installation of electric vehicle supply equipment to provide electric vehicle charging stations at such time as it is needed for use by residents? • Multiple-family projects of more than 17 dwelling units: Of the total required listed cabinets, boxes or enclosures, would 50% have the necessary electric vehicle supply equipment installed to provide active electric vehicle charging stations ready for use by residents? • Non-residential projects: Of the total required listed cabinets, boxes or enclosures, would 50% have the necessary electric vehicle supply equipment installed to provide active electric vehicle charging stations ready for use? Check "N/A" only if the project is a single-family project or would not require the provision of listed cabinets, boxes, or enclosures connected to a conduit linking the parking spaces with electrical service, e.g., projects requiring fewer than 10 parking spaces.	☐	☐	☑

Strategy 3: Bicycling, Walking, Transit & Land Use (Complete this section if project includes non-residential or mixed uses)			
4. *Bicycle Parking Spaces* Would the project provide more short- and long-term bicycle parking spaces than required in the City's Municipal Code (Chapter 14, Article 2, Division 5)?[6] Check "N/A" only if the project is a residential project.	☐	☐	☑

[6] Non-portable bicycle corrals within 600 feet of project frontage can be counted towards the project's bicycle parking requirements.

5. *Shower facilities*

If the project includes nonresidential development that would accommodate over 10 tenant occupants (employees), would the project include changing/shower facilities in accordance with the voluntary measures under the California Green Building Standards Code as shown in the table below?

Number of Tenant Occupants (Employees)	Shower/Changing Facilities Required	Two-Tier (12" X 15" X 72") Personal Effects Lockers Required
0-10	0	0
11-50	1 shower stall	2
51-100	1 shower stall	3
101-200	1 shower stall	4
Over 200	1 shower stall plus 1 additional shower stall for each 200 additional tenant-occupants	1 two-tier locker plus 1 two-tier locker for each 50 additional tenant-occupants

☐ ☐ ☑

Check "N/A" only if the project is a residential project, or if it does not include nonresidential development that would accommodate over 10 tenant occupants (employees).

6. *Designated Parking Spaces*

If the project includes a nonresidential use in a TPA, would the project provide designated parking for a combination of low-emitting, fuel-efficient, and carpool/vanpool vehicles in accordance with the following table?

Number of Required Parking Spaces	Number of Designated Parking Spaces
0-9	0
10-25	2
26-50	4
51-75	6
76-100	9
101-150	11
151-200	18
201 and over	At least 10% of total

This measure does not cover electric vehicles. See Question 4 for electric vehicle parking requirements. ☐ ☐ ☑

Note: Vehicles bearing Clean Air Vehicle stickers from expired HOV lane programs may be considered eligible for designated parking spaces. The required designated parking spaces are to be provided within the overall minimum parking requirement, not in addition to it.

Check "N/A" only if the project is a residential project, or if it does not include nonresidential use in a TPA.

7. *Transportation Demand Management Program* If the project would accommodate over 50 tenant-occupants (employees), would it include a transportation demand management program that would be applicable to existing tenants and future tenants that includes: At least one of the following components: • Parking cash out program • Parking management plan that includes charging employees market-rate for single-occupancy vehicle parking and providing reserved, discounted, or free spaces for registered carpools or vanpools • Unbundled parking whereby parking spaces would be leased or sold separately from the rental or purchase fees for the development for the life of the development And at least three of the following components: • Commitment to maintaining an employer network in the SANDAG iCommute program and promoting its RideMatcher service to tenants/employees • On-site carsharing vehicle(s) or bikesharing • Flexible or alternative work hours • Telework program • Transit, carpool, and vanpool subsidies • Pre-tax deduction for transit or vanpool fares and bicycle commute costs • Access to services that reduce the need to drive, such as cafes, commercial stores, banks, post offices, restaurants, gyms, or childcare, either onsite or within 1,320 feet (1/4 mile) of the structure/use? Check "N/A" only if the project is a residential project or if it would not accommodate over 50 tenant-occupants (employees).	☐	☐	☑

Step 3: Project CAP Conformance Evaluation (if applicable)

The third step of the CAP consistency review only applies if Step 1 is answered in the affirmative under option B. The purpose of this step is to determine whether a project that is located in a TPA but that includes a land use plan and/or zoning designation amendment is nevertheless consistent with the assumptions in the CAP because it would implement CAP Strategy 3 actions. In general, a project that would result in a reduction in density inside a TPA would not be consistent with Strategy 3.The following questions must each be answered in the affirmative and fully explained.

1. **Would the proposed project implement the General Plan's City of Villages strategy in an identified Transit Priority Area (TPA) that will result in an increase in the capacity for transit-supportive residential and/or employment densities?**

   Considerations for this question:
   - Does the proposed land use and zoning designation associated with the project provide capacity for transit-supportive residential densities within the TPA?
   - Is the project site suitable to accommodate mixed-use village development, as defined in the General Plan, within the TPA?
   - Does the land use and zoning associated with the project increase the capacity for transit-supportive employment intensities within the TPA?

2. **Would the proposed project implement the General Plan's Mobility Element in Transit Priority Areas to increase the use of transit?**

   Considerations for this question:
   - Does the proposed project support/incorporate identified transit routes and stops/stations?
   - Does the project include transit priority measures?

3. **Would the proposed project implement pedestrian improvements in Transit Priority Areas to increase walking opportunities?**

   Considerations for this question:
   - Does the proposed project circulation system provide multiple and direct pedestrian connections and accessibility to local activity centers (such as transit stations, schools, shopping centers, and libraries)?
   - Does the proposed project urban design include features for walkability to promote a transit supportive environment?

4. **Would the proposed project implement the City of San Diego's Bicycle Master Plan to increase bicycling opportunities?**

   Considerations for this question:
   - Does the proposed project circulation system include bicycle improvements consistent with the Bicycle Master Plan?
   - Does the overall project circulation system provide a balanced, multimodal, "complete streets" approach to accommodate mobility needs of all users?

5. **Would the proposed project incorporate implementation mechanisms that support Transit Oriented Development?**

   Considerations for this question:
   - Does the proposed project include new or expanded urban public spaces such as plazas, pocket parks, or urban greens in the TPA?
   - Does the land use and zoning associated with the proposed project increase the potential for jobs within the TPA?
   - Do the zoning/implementing regulations associated with the proposed project support the efficient use of parking through mechanisms such as: shared parking, parking districts, unbundled parking, reduced parking, paid or time-limited parking, etc.?

6. **Would the proposed project implement the Urban Forest Management Plan to increase urban tree canopy coverage?**

   Considerations for this question:
   - Does the proposed project provide at least three different species for the primary, secondary and accent trees in order to accommodate varying parkway widths?
   - Does the proposed project include policies or strategies for preserving existing trees?
   - Does the proposed project incorporate tree planting that will contribute to the City's 20% urban canopy tree coverage goal?

CLIMATE ACTION PLAN CONSISTENCY CHECKLIST
ATTACHMENT A

This attachment provides performance standards for applicable Climate Action Pan (CAP) Consistency Checklist measures.

Table 1 Roof Design Values for Question 1: Cool/ Green Roofs supporting Strategy 1: Energy & Water Efficient Buildings of the Climate Action Plan

Land Use Type	Roof Slope	Minimum 3-Year Aged Solar Reflectance	Thermal Emittance	Solar Reflective Index
Low-Rise Residential	≤ 2:12	0.55	0.75	64
	> 2:12	0.20	0.75	16
High-Rise Residential Buildings, Hotels and Motels	≤ 2:12	0.55	0.75	64
	> 2:12	0.20	0.75	16
Non-Residential	≤ 2:12	0.55	0.75	64
	> 2:12	0.20	0.75	16

Source: Adapted from the California Green Building Standards Code (CALGreen) Tier 1 residential and non-residential voluntary measures shown in Tables A4.106.5.1 and A5.106.11.2.2, respectively. Roof installation and verification shall occur in accordance with the CALGreen Code.

CALGreen does not include recommended values for low-rise residential buildings with roof slopes of ≤ 2:12 for San Diego's climate zones (7 and 10). Therefore, the values for climate zone 15 that covers Imperial County are adapted here.

Solar Reflectance Index (SRI) equal to or greater than the values specified in this table may be used as an alternative to compliance with the aged solar reflectance values and thermal emittance.

Table 2 Fixture Flow Rates for Non-Residential Buildings related to Question 2: Plumbing Fixtures and Fittings supporting Strategy 1: Energy & Water Efficient Buildings of the Climate Action Plan

Fixture Type	Maximum Flow Rate
Showerheads	1.8 gpm @ 80 psi
Lavatory Faucets	0.35 gpm @60 psi
Kitchen Faucets	1.6 gpm @ 60 psi
Wash Fountains	1.6 [rim space(in.)/20 gpm @ 60 psi]
Metering Faucets	0.18 gallons/cycle
Metering Faucets for Wash Fountains	0.18 [rim space(in.)/20 gpm @ 60 psi]
Gravity Tank-type Water Closets	1.12 gallons/flush
Flushometer Tank Water Closets	1.12 gallons/flush
Flushometer Valve Water Closets	1.12 gallons/flush
Electromechanical Hydraulic Water Closets	1.12 gallons/flush
Urinals	0.5 gallons/flush

Source: Adapted from the California Green Building Standards Code (CALGreen) Tier 1 non-residential voluntary measures shown in Tables A5.303.2.3.1 and A5.106.11.2.2, respectively. See the California Plumbing Code for definitions of each fixture type.

Where complying faucets are unavailable, aerators rated at 0.35 gpm or other means may be used to achieve reduction.

Acronyms:
gpm = gallons per minute
psi = pounds per square inch (unit of pressure)
in. = inch

Table 3 Standards for Appliances and Fixtures for Commercial Application related to Question 2: Plumbing Fixtures and Fittings supporting Strategy 1: Energy & Water Efficient Buildings of the Climate Action Plan

Appliance/Fixture Type	Standard	
Clothes Washers	Maximum Water Factor (WF) that will reduce the use of water by 10 percent below the California Energy Commissions' WF standards for commercial clothes washers located in Title 20 of the *California Code of Regulations*.	
Conveyor-type Dishwashers	0.70 maximum gallons per rack (2.6 L) (High-Temperature)	0.62 maximum gallons per rack (4.4 L) (Chemical)
Door-type Dishwashers	0.95 maximum gallons per rack (3.6 L) (High-Temperature)	1.16 maximum gallons per rack (2.6 L) (Chemical)
Undercounter-type Dishwashers	0.90 maximum gallons per rack (3.4 L) (High-Temperature)	0.98 maximum gallons per rack (3.7 L) (Chemical)
Combination Ovens	Consume no more than 10 gallons per hour (38 L/h) in the full operational mode.	
Commercial Pre-rinse Spray Valves (manufactured on or after January 1, 2006)	Function at equal to or less than 1.6 gallons per minute (0.10 L/s) at 60 psi (414 kPa) and • Be capable of cleaning 60 plates in an average time of not more than 30 seconds per plate. • Be equipped with an integral automatic shutoff. • Operate at static pressure of at least 30 psi (207 kPa) when designed for a flow rate of 1.3 gallons per minute (0.08 L/s) or less.	

Source: Adapted from the California Green Building Standards Code (CALGreen) Tier 1 non-residential voluntary measures shown in Section A5.303.3. See the California Plumbing Code for definitions of each appliance/fixture type.

Acronyms:
L = liter
L/h = liters per hour
L/s = liters per second
psi = pounds per square inch (unit of pressure)
kPa = kilopascal (unit of pressure)

UPDATE GEOTECHNICAL INVESTIGATION
AND BASIS OF DESIGN
8230 PRESTWICK DRIVE
LA JOLLA, CALIFORNIA

Prepared for
BLUE HERON
Las Vegas, Nevada

TerraCosta
Consulting Group

Prepared by
TERRACOSTA CONSULTING GROUP, INC.
San Diego, California

Project No. 3023
July 12, 2018

TerraCosta Consulting Group

Geotechnical Engineering
Coastal Engineering
Maritime Engineering

Project No. 3023
July 12, 2018

Ms. Amy Finchem
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4675 W Tico Ave., Suite 1115
Las Vegas, Nevada 89118

Via email: afinchem@blueheron.com

UPDATE GEOTECHNICAL INVESTIGATION
AND BASIS OF DESIGN
8230 PRESTWICK DRIVE
LA JOLLA, CALIFORNIA

Dear Ms. Finchem:

In accordance with the request of Ms. Chandra Slaven and our Proposal No. 16132 dated May 7, 2018, we have performed an update geotechnical investigation and basis of design study for the proposed single-family residential project located at 8230 Prestwick Drive in the community of La Jolla, City of San Diego, California.

This update report presents the results of our findings, our geologic and engineering analyses of subsurface conditions at the site, and our conclusions and recommendations pertaining to the geotechnical aspects of site development. The original "Report of Preliminary Geotechnical Investigation" for the project, dated November 30, 2016, was prepared by Christian Wheeler Engineering (copy included in Appendix A).

We agree with the geologic and geotechnical findings and recommendations in the appended Christian Wheeler report, except as noted in this report. TerraCosta Consulting Group takes responsibility as the geotechnical engineer-of-record for the subject project.

3890 Murphy Canyon Road, Suite 200 ▲ San Diego, California 92123 ▲ (858) 573-6900 *voice* ▲ (858) 573-8900 *fax*
www.terracosta.com

We appreciate the opportunity to be of service and trust this information meets your needs. If you have any questions or require additional information, please give us a call.

Very truly yours,
TERRACOSTA CONSULTING GROUP, INC.

Walter F. Crampton, Principal Engineer
R.C.E. 23792, R.G.E. 245

Braven R. Smillie, Principal Geologist
C.E.G. 207, P.G. 402

WFC/ak
Attachments

cc: Chandra Slaven, cslaven@blueheron.com

TABLE OF CONTENTS

UPDATE GEOTECHNICAL INVESTIGATION
AND BASIS OF DESIGN
8230 PRESTWICK DRIVE
LA JOLLA, CALIFORNIA

1 INTRODUCTION & PROJECT DESCRIPTION

The subject property is located on the westerly side of Prestwick Drive, above La Jolla Shores, on the Pacific shoreline at 8230 Prestwick Drive in La Jolla, California. The Vicinity Map (Figure 1) shows the project site in the context of regional topographic and cultural features, and the Regional Geology Map (Figure 1a) illustrates local area geology at the same scale. The Site Plan (Figure 2) and the Generalized Geologic Cross Sections (Figure Nos. 3, 4, 5, & 6) summarize existing topographic and geologic conditions at the site, as well as the proposed lower level and the extent of the grading and construction footprint. Figure 7 presents the proposed lower level foundation plan.

Current architectural plans indicate that, following demolition of the existing residence, a new single-story single-family residence will be constructed with 5,213 square feet at the main level (above grade), 5,537 square feet at the lower level (basement), and 1,422 square feet of decking, along with a pool that will be structurally attached to the main residential structure. Approximate setbacks for the new structure are as follows:

- Front setback from east property line – 15 to 16 feet;
- Top-of-slope setback – approximately 18 feet; and
- North and south side-yard setbacks – 6 feet.

2 PURPOSE & SCOPE OF WORK

The purpose of our update geotechnical investigation is to provide information to assist you and your consultants in project design, and to address City of San Diego and La Jolla Town Council concerns regarding the geotechnical aspects of the project.

In particular, our investigation is designed to address the following geologic and geotechnical site conditions in relation to the proposed project:

- The geologic setting of the site;
- Subsurface soil conditions;
- Groundwater;
- Potential geologic hazards;
- Foundation design, including allowable soil bearing and earth pressure values;
- Soil creep loads to be resisted by the drilled pier and grade beam foundation system; and
- Slope stability.

3 SITE CONDITIONS AND GEOLOGY

At approximate elevation 325 feet (MSLD), 32.856° North Latitude and 117.247° West Longitude, the property is located at the top of a north-south trending ridge above the 2,500± foot-wide coastal terrace at La Jolla Shores beach in the community of La Jolla, California.

We understand that grading for the Prestwick Estates Subdivision took place circa 1961. The original grading resulted in an approximately 100 foot by 100 foot buildable cut/fill lot pad at 8230 Prestwick Drive and a westerly descending fill-over-natural-and-cut slope inclined at approximately 1.5 to 1 (horizontal to vertical). Finally, as illustrated on the Site Plan (Figure 2) and Cross Sections (Figures 3 through 6), the current project plans include excavating for a lower level, down to elevation 309.5 feet.

3.1 Subsurface Soil Conditions

Two soil and geologic units exist within the general project site area as described below.

Ardath Shale (Ta): The Ardath Shale or "Ardath Formation" is typically described as a middle Eocene-age (40 to 50 million years old) weakly fissile or fine-bedded olive-gray silty to clayey shale. The Report of Preliminary Geotechnical Investigation by Christian Wheeler dated November 30, 2016 (Appendix A) describes the Ardath strata as generally consisting of light gray to yellowish-brown, moist, hard, clayey silt-silty clay (ML-CL) and silty clay (CL).

Artificial Fill Soils (Qaf): Fill soils derived locally from the Ardath Shale, and apparently placed during the 1961 grading for the Prestwick Estates development project, were reported by Christian Wheeler to have exhibited an expansion index ranging between 51 and 90 (moderately to very highly expansive).

4 GEOLOGIC/GEOTECHNICAL HAZARDS

4.1 Faulting and Seismicity

The site is located at 32.856° North Latitude and 117.247° West Longitude in a moderately active seismic region of Southern California that is subject to significant hazards from moderate to large earthquakes. Ground shaking from 10 major active fault zones could affect the site in the event of an earthquake. The nearest of these, the Rose Canyon fault zone, has been mapped approximately 2,800 feet southwest of the site where it trends offshore, ultimately becoming part of the Newport-Inglewood/Rose Canyon fault system. No known active faults have been mapped, nor were any noted during our geologic/geotechnical evaluation, at or in the immediate vicinity of the site.

4.1.1 *Ground Surface Rupture*

We agree with the Christian Wheeler opinion that since no known active faults traverse the subject site, the risk for ground surface rupture is low.

4.1.2 *Ground Shaking*

Because of its proximity to the active Rose Canyon fault zone, the risk to the site from ground shaking is high. Using the computer program EQFAULT and a Soil Class D, we estimate that peak ground accelerations at the site will be on the order of 0.581g from an earthquake produced on the Rose Canyon fault zone located about 0.53 mile to the southwest.

4.2 Landslides

As an integral part of our geotechnical investigation for this update report project, we reviewed the following documents:

- City of San Diego Seismic Safety Study (Geologic Hazards and Faults);
- 1953 USDA San Diego County Stereopair Aerial Photograph Nos. AXN-4M-86 and 87, as well as AXN-8M-2 and 3; and
- The November 30, 2016, Report of Preliminary Geotechnical Investigation prepared by Christian Wheeler Engineering for the subject project (Appendix A).

Our investigation did not reveal the presence of any landslides on the site. No landslides have been mapped as being present, either on or immediately adjacent to the site. Our review of the 1953 stereopair aerial photographs (before the development of Prestwick Estates) provided no indications of landslides in the area of the project site. Finally, our review of the San Diego Seismic Safety Study assigns the project site area to "Geologic Hazard Category 26 – Ardath: Unfavorable Geologic Structure." However, our review of geologic and geotechnical studies covering the project site area and surrounding westerly facing slopes indicates the Ardath Shale strata to be inclined between 5° and 7° to the northeast, with an average apparent dip of 4° into the slope below the project site, thus classifying it as not adverse to the slope and consequently not exhibiting "unfavorable" geologic structure.

5 GROUNDWATER

As reported on Page 5 of Christian Wheeler's preliminary geotechnical report (Appendix A) "*minor seepage was encountered in Boring B-3 at the contact between the artificial fill and the Ardath Shale; however, similar groundwater conditions were not observed in the other two borings or the test pit. We do not anticipate any significant groundwater related conditions during or after proposed construction. However, it should be recognized that minor groundwater seepage problems might occur after construction and landscaping are completed, even at a site where none were present before construction. These are usually minor phenomena and are often a result of an alteration in drainage patterns and/or an increase in irrigation water.*"

While we agree with the above statement, we strongly advise and emphasize that, because of the potential for groundwater to create instability and settlement in fill soils derived from the Ardath Shale, all grading and landscaping should be designed to reduce the potential for surface water infiltration into the wedge of fill soils that underlies the upper half of the steeply inclined, westerly facing slope on the property.

6 SLOPE STABILITY

Although our slope stability analyses indicate the formational soils that comprise the 40 to 50 million year old Ardath Shale strata, which underlie the 65± foot-high 1.5:1 (horizontal to vertical) slope on the property to be grossly stable, the artificial fill soils derived from the Ardath Shale are highly prone to lateral fill extension or downslope soil creep, thus resulting in a higher than average risk of differential settlement, slope creep, and the resulting damage to settlement sensitive structures. The reasons for this include the following:

- Numerous soil investigations and geotechnical studies over the 57 years since the original grading for the Prestwick Estates have found that compacted fill soils generally fall below the standards of the current City Grading Ordinance;

- The very steep slopes (both cut and fill slopes) around Prestwick Estates are no longer permitted by City codes;

- The City Grading Ordinance-required "benching" of natural slopes in preparation for the placement of fill soils in some areas around Prestwick Estates has been found to be inadequate, thus leaving weak and weathered clayey overburden soils in place and allowing a zone of weakness between the fill soils and the underlying formational soils; and

- The clayey fill soils derived from the Ardath Shale are known to be prone to lateral fill extension or downslope soil creep, and although many individual cases have been mitigated throughout the project, it is well known in the geotechnical community that the development has a history of such issues, some examples of which have been very serious and expensive to repair. A minor example of this slope creep phenomenon is the pool decking at the site, which appears to be cracking and settling differentially as the underlying soils move downslope over time.

The soil strengths used in our analyses are summarized below:

Material	Total Unit Weight pcf (γc)	Cohesion psf (c)	Angel of Friction Degrees (φ)
Artificial Fill Soils	110	100	30
Ardath Shale	130	600	30

7 DOWNSLOPE SOIL CREEP

In general, soil creep is soil movement that continues under constant stress conditions. Within the literature, soil creep is considered either constant under a given set of conditions or ever decreasing under constant conditions. As such, creep continues. For illustrative purposes, if one assumes a constant creep rate model, the rate of soil creep can be conceptualized as being inversely related to shear stress and soil stiffness. Hence, an increase in the rate of soil creep would be anticipated with either an increase in applied shear stress or a decrease in soil stiffness. Likewise, if one assumes a decreasing soil creep model under constant stress conditions, the rate of change of creep rate would also be inversely related to shear stress and soil stiffness. Thus, a decrease in the change in rate of creep should be anticipated with either a decrease in shear stress or an increase in soil stiffness.

Fill soils comprised of the Ardath Shale constructed in the early 1960s were compacted under the ASTM D-698 test standard for maximum laboratory compaction, which uses a three-layer 4-inch-diameter mold, with each layer of soil compacted with a 5½-pound hammer with a 12-inch drop receiving 25 blows per layer, which results in a 12,400 foot-pound compactive effort for the laboratory standard. In contrast, the current laboratory standard is ASTM D-1557, which with a 4-inch mold compacts the soil in five layers with a 10-pound hammer and an 18-inch drop with 25 blows per layer, resulting in a laboratory compaction standard of 56,000 foot-pounds of energy. In 1961, the laboratory compaction standard was ASTM D-698 (also referred to as the Standard Proctor Test), with 90 percent relative compaction when using D-698 equivalent to about 83 to 84 percent relative compaction when using ASTM D-1557. Thus, all of the compacted fills within the Prestwick Estates development were compacted to a relative compaction in the low 80 percent range when compared to contemporary laboratory standards.

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The Eocene-age Ardath Shale exhibits some rather unusual engineering properties when recompacted as a fill soil. In its natural condition, this very hard formational shale, when excavated from a cut for use in compacted fills, comes out of the ground as small, partially crushed, rock-like angular fragments that, to a certain extent, still retain their rock-like structure when compacted. Due to this fact, some additional pore space exists around the rock fragments when compacted in the laboratory to the ASTM D-698 or Proctor standard, thus resulting in relatively low laboratory maximum densities for use in subsequently calculating the relative compaction during the placement of engineered fill soils. This same rock fragment material, a decade or so later, tends to break down into a softer, clayey structure that, when compacted again in the laboratory to the ASTM D-698 standard, results in higher laboratory maximum densities, primarily due to the weathering of the crushed rock fragments over the course of a decade or more.

Coupling these with the often less-than-perfect benching practices of the 1960s has resulted in numerous cases of damaging downslope soil creep in projects developed on the Ardath Shale.

This downslope creep, when encountering a drilled pier, tends to at least partially flow around the pier, imparting high lateral earth pressures approaching the full passive earth pressures characteristic of this compacted fill material. Our firm has observed drilled pier foundations that, decades after construction, have yielded and rotated due to these surprisingly high lateral earth pressures.

Accordingly, in an abundance of caution, we have assumed worst-case scenarios for earth pressures applied to drilled piers supporting the structural lower-level floor.

8 FOUNDATION DESIGN

The proposed structure is approximately 90 feet in square dimension, with a lower-level finish floor elevation of 309.54 feet, requiring temporary construction excavations from 13 to 15 feet below grade. The Site Plan (Figure 2) shows the cut/fill line through the existing single-level at-grade residence, along with the more westerly cut/fill line created by the proposed lower-level excavation. That lower-level cut/fill line removes a substantial portion of the undesirable expansive and creep-sensitive fill soils, with the westerly approximately 35 percent of the proposed improvements still overlying these unsuitable fill soils. In order to mitigate relatively high lateral loads from downslope creep within this side-hill fill, we have provided lateral design loads to fully restrain the adverse effects of downslope creep, in part restrained by sufficient embedment into the underlying formational soils and through additional lateral restraint provided at the top of the drilled pier through the east-west grade beams supporting the rest of the structure. Lateral restraint through the grade beams is most efficiently provided through the drilled pier foundations supporting the eastern portion of the structure on cut, yet still expansive, Ardath Shale. While it could be argued that the drilled piers on the eastern portion of the structure are unnecessary and the north-south grade beams could be deepened sufficient to provide the required additional lateral capacity, the additional drilled pier foundations substantially mitigate, if not eliminate, foundation distress associated with expansive clay soils and soil creep.

Architectural Sheet A2.21 has been reproduced in this report as Figure 7, on which all of the drilled piers are shown, with Piers 1 through 14 subjected to additional creep loads, with the required lateral restraint to be provided by the east-west grade beams listed in Table 1. Also provided in Table is the estimated depth to the underlying bedrock at each pier location measured below the bottom of the grade beam, along with the required minimum embedment depth into formational soils. Down-drag loads for the 14 drilled piers are also tabulated on Table 1. Calculations are also provided in Appendix B.

8.1 Building Foundations

From discussions with the design team, we understand that the entire structure will be supported on drilled piers tied together with grade beams supporting a lower floor slab. The east-west grade beams are to be designed to accommodate the additional required lateral capacity listed in Table 1 and restrained by those drilled piers located a minimum of 10 feet easterly of the lower-level cut/fill line shown on Figure 7. The available lateral capacity of drilled piers easterly of the lateral setback line is provided below in Section 9.1 – Lateral Pier Capacity.

8.2 Drilled Pier Design Criteria

Drilled pier foundation design is typically based on shaft friction and end bearing. However, end bearing is typically excluded in the analysis, unless the condition of the bottom of the drilled pier shaft can be verified. The inspection of the bottom of a 2-foot-diameter drilled shaft is somewhat problematic, but with some effort can be inspected. That said, the working or mobilized load of a drilled pier is a function of the settlement of the concrete pier with both skin friction and end bearing. The estimated settlement for full skin friction mobilized is about 0.8 percent of the shaft diameter, which for a 24-inch-diameter drilled pier is approximately 0.2 inch. The full mobilization of the end bearing capacity of a drilled pier is taken as the settlement corresponding to about 5 percent of the shaft diameter. The bearing capacity of the drilled pier can be assumed to mobilize linearly for each component of the capacity up to the fully mobilized value, where it is assumed to become constant. Thus, developing the ultimately end bearing capacity of the drilled pier would require approximately 1.2 inches of settlement (0.05 x 24”), which, if reduced to correspond with the ultimate skin friction, would require a factor of safety of about 6.25 against ultimate end bearing, which in most instances would discourage the use of end bearing. Accordingly, we recommend that all drilled piers be designed for an allowable skin friction of 800 psf per foot of embedment into formational soils. Minimum embedment for all piers drilled into the intact formational Ardath Shale is 10 feet. Down-drag loads for the 14 drilled piers listed in Table 1 are also included and must be added to the design axial load for any of these drilled piers when calculating the required embedment into formational soils.

As indicated above, we anticipate total settlements of drilled pier foundations to be on the order of 0.2 inch, with differential settlements between adjacent drilled pier foundations of 0.15 inch.

8.3 Structural Mat Foundation

All of the grade beams for the structural mat foundation easterly of the lower level cut/fill line shown on Figure 7 will be embedded in footing excavations extended into Ardath Shale, with additional bearing capacity available if necessary. However, we recommend that all foundation loads be supported by the proposed drilled pier foundations, in part to minimize differential settlements between adjacent foundation elements. Accordingly, there is no need to clean the bottoms of grade beam excavations, other than to ensure that the excavation provides the minimum structural dimensions for the grade beams.

It is anticipated that the fill soils supporting grade beams westerly of the lower level cut/fill line shown on Figure 7 will settle over the life of the structure, with footing excavations made in these fill soils only facilitating the initial construction of the grade beams.

We suggest that the structural slab spanning the adjacent grade beams shown on Figure 7 be placed on a minimum 3-inch-thick foam mat across both the cut and fill portions of the building pad to eliminate potential heave forces from the expansive soils that might otherwise dome, and worst case crack, the structural floor slab.

8.4 Seismic Design Parameters per CBC

For the proposed structure, design for earthquake loads per Section 1613 of the California Building Code (CBC, 2016 Edition), Title 24, we have revised slightly the seismic design factors tabulated on Page 18 of Christian Wheeler's report to reflect the 2016 CBC. The updated seismic design parameters follow:

SEISMIC DESIGN FACTORS

Site Coordinates: Latitude Longitude	32.8559 -117.2475
Site Class	D
Site Coefficient F_a	1.0
Site Coefficient F_v	1.5
Spectral Response Acceleration at Short Period S_S	1.302 g
Spectral Response Acceleration at 1 Second Period S_1	0.505 g
$S_{MS} = F_aS_s$	1.300 g
$S_{MS} = F_vS_1$	0.757 g
$S_{DS} = 2/3*S_{MS}$	0.867 g
$S_{DI} = 2/3*S_{M1}$	0.505 g

9 DRILLED PIER WALL DESIGN ALTERNATIVE

As discussed with members of the design team, in certain instances, and in particular the proposed property line walls at both the northwest and southwest corners of the proposed structure (as depicted on Sheet A1.02), a cantilevered drilled pier wall may facilitate the construction of property line walls, or in other instances where a permanent cantilevered wall without a footing may be desirable. An example of a drilled pier wall is provided on Figure 8.

Vertical drilled pier walls may be designed as either cantilevered or tied-back structures. Wall loads increase roughly with the square of the unsupported height, and cantilevered walls are typically limited to unsupported wall heights on the order of 15 feet.

Vertical drilled pier walls are loaded by the active earth pressure (including any surcharge loads) behind the wall. Resistance to overturning is developed through deflection in the wall, which mobilizes the reaction of the soil into which the wall is embedded. The resisting pressure applied by the soil to a drilled pier wall depends upon the relative stiffness of the pier and soil, as well as the depth of embedment.

If sufficient embedment is not available, overturning forces must then be resisted by a tied-back system utilizing tie rods attached to concrete anchors some distance behind the wall. Cantilevered vertical walls are usually less expensive than tied-back walls and are easier to construct. One disadvantage, however, is that, as with conventional cantilevered walls, a certain amount of post-construction deflection is required to fully mobilize the strength of the soil fronting the wall. This occurs with all cantilevered walls, including all CMU walls and Caltrans-type reinforced concrete walls. Actual wall deflection is a function of the active earth pressure loading the wall and the stiffness of the wall system.

Failure of a laterally loaded pier takes place either when the maximum bending moment in the loaded pier reaches the ultimate or yield resistance of the pier section, or when the lateral earth pressures reach the ultimate lateral resistance of the soil along the total length of the pier. For purposes of definition, failure of piers with relatively "short embedment" takes place when the pier rotates as a unit with respect to a point located close to its toe. Failures of piers with relatively "long embedment" occur when the maximum bending moment applied to the pier exceeds the yield resistance of the pier section and a plastic hinge forms at the section of maximum bending moment. Investigators have suggested that piers be grouped relative to their dimensionless depth of embedment, L/T, where:

L = embedment length of the pier in feet, and

$$T = \left(\frac{EI}{f}\right)^{\frac{1}{5}} \text{ (divided by 12 to convert inches to feet)}$$

The quantity EI is the stiffness of the pier section, and f (coefficient of variation of soil modulus) for the sloping formational soils would be on the order of 60 pounds per cubic inch, and on the order of 100 pounds per cubic inch for the easterly pier-supported grade beams. Short piles are generally defined as L/T being less than 2.0, and long piers are generally defined as L/T being larger than 4.0. Thus, minimum pier embedment was selected based on an L/T of 2 to 3, depending upon loading conditions and required lateral capacity.

In order to determine the structural requirements for both an alternative property line drilled pier wall and the drilled pier-supported building foundations, we have evaluated the soil-induced moment, shear, and deflection of drilled piers using the elastic theory approach developed by Matlock and Reese (1962). A condensed version of this approach is outlined in the NAVFAC Design Manual DM-7.2, Chapter 5, Section 7. Calculations are also provided in Appendix B.

9.1 Lateral Pier Capacity

As an illustrative example, if the property line walls are constructed as a drilled pier wall with 2-foot-diameter drilled piers on 6-foot centers resisting a 40 pound per cubic foot equivalent fluid pressure, post-construction top-of-wall deflections would be approximately 1/4 inch for an 8-foot-high cantilevered drilled pier wall.

For drilled piers supporting grade beams along the eastern portion of the building pad designed to resist lateral loads developed by the more westerly drilled piers in Table 1, when using a Matlock and Reese solution for a Case II condition (assuming pier fixity within the grade beam), 10-foot-deep drilled piers designed to resist a 30-kip lateral load have a soil-induced moment of 137 kip-feet with a ground surface deflection of 0.11 inch (see attached calculations). In summary, drilled pier walls and drilled pier foundations provide for considerable flexibility in foundation design.

10 CONSTRUCTION CUTS AND EXCAVATIONS

We recommend that construction cuts and excavations comply with Cal OSHA and OSHA recommendations and guidelines. On Page 20 of Christian Wheeler's report, they recommend that temporary shoring be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot, which we would also recommend for the more westerly side-hill fill soils. Temporary shoring in intact Ardath Shale can be designed to resist an equivalent fluid pressure of 20 pounds per cubic foot. Please refer to Cross Sections 1 and 3 in Figures 3 and 5, respectively, which show the approximate extent of intact Ardath Shale on the north (Cross Section 1) and south (Cross Section 3) sides of the proposed construction excavation. Temporary shoring designed to resist the Ardath Shale (Ta) should be designed for an equivalent fluid pressure of 20 pcf. That portion of the temporary shoring designed to resist the more westerly sloping side-hill fill (Qaf) should be designed to resist an equivalent fluid pressure of 40 pcf. For those portions of the temporary shoring supporting fill over cut, the shoring should be designed for 40 pcf for the fill portion restrained by the shoring, and 20 pcf for the formational portion of the shored excavation.

The easterly part of the excavation, which exposes hard intact Ardath Shale will stand in temporary construction excavations at 1/2:1 (horizontal to vertical). The upper 2 feet of material exposed in Christian Wheeler’s test pit adjacent to Prestwick Drive will require a minimum construction cut excavation of no steeper than 45 degrees, or 1:1. Moreover, the top of the excavation should be no closer than 5 feet from any existing improvements or construction equipment. At least in the vicinity of the southerly garage, construction-period shoring may be eliminated.

11 LIMITATIONS

Geotechnical engineering and the earth sciences are characterized by uncertainty. Professional judgments presented herein are based partly on our evaluation of the technical information gathered, partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgments rendered meet the current professional standards. We do not guarantee the performance of the project in any respect.

We have evaluated only a small portion of the pertinent soil and groundwater conditions at the subject site. The opinions and conclusions made herein are based on the assumption that those subsurface soil conditions do not deviate appreciably from those encountered during the November 2016 Christian Wheeler field investigation. We recommend that technical staff personnel from our office observe grading and construction to assist in identifying any soil conditions that may differ significantly from those encountered during that investigation. Additional recommendations may be required at that time.

REFERENCES

1. Blake, T.F., 2000, EQFAULT, a computer program for deterministic prediction of peak horizontal acceleration, Computer Services and Software.

2. Christian Wheeler Engineering, November 30, 2016, Report of Preliminary Geotechnical Investigation, Proposed Hershfield Residence, 8230 Prestwick Drive, La Jolla, California.

3. City of San Diego, 2008, Seismic Safety Study (Geologic Hazards and Faults).

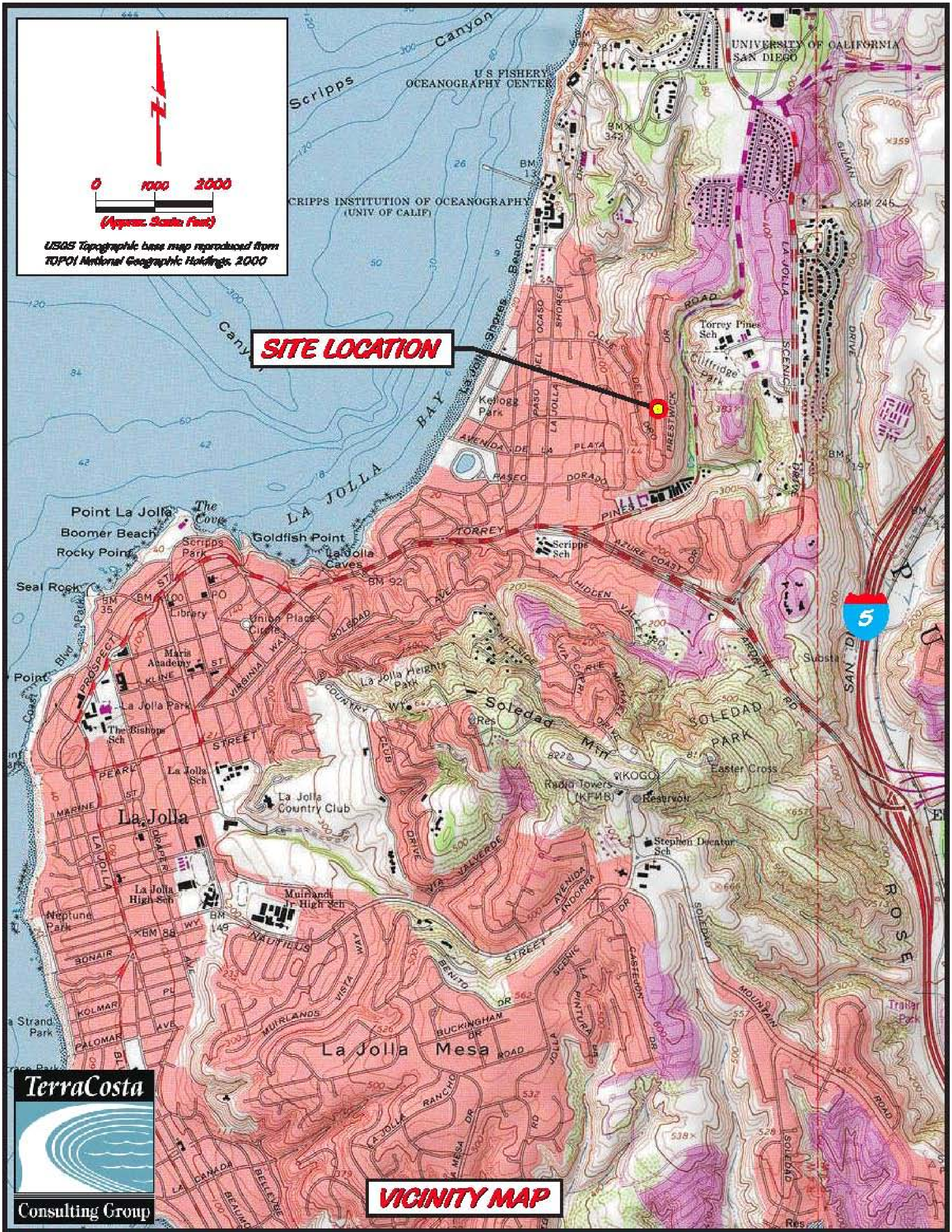
4. Kennedy, M.P., S.S. Tan, 2008, Geologic Map of the San Diego 30' x 60' Quadrangle, California.

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

6. USDA, 1953, San Diego County Stereopair Aerial Photograph Nos. AXN-4M-86 and 87, AXN-8M-2 and 3.

TABLE 1
DRILLED PIER DESIGN CRITERIA

Drilled Pier No.	Fill Depth, Ft	Required Lateral Restraint, Kips	Minimum Embedment into Ta, ft	Drag-Down Loads, Kips
1	9	26.5	20	31.7
2	10	30.0	20	36.8
3	10	30.0	20	36.8
4	9	26.5	20	31.7
5	10	31.3	20	36.8
6	8	26.0	20	26.6
7	8	26.0	20	26.6
8	4	9.5	15	10.6
9	5	13.4	15	14.1
10	5	13.4	15	14.1
11	4	9.5	15	10.6
12	3	6.1	15	7.5
13	2	4.0	15	4.6
14	2	4.0	15	4.6



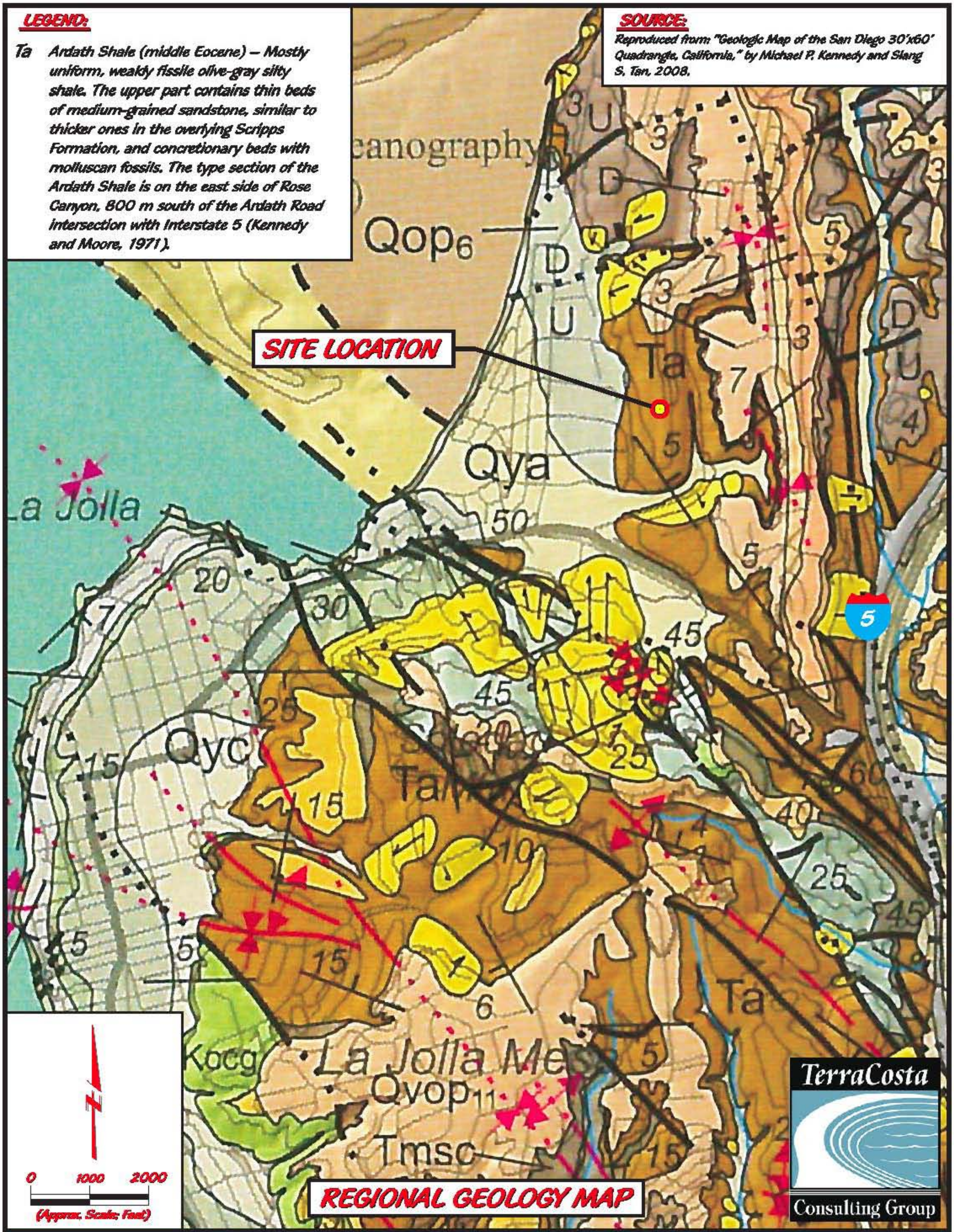
Project: 8230 Prestwick Drive **Project No. 3023** **Figure No. 1**

LEGEND:

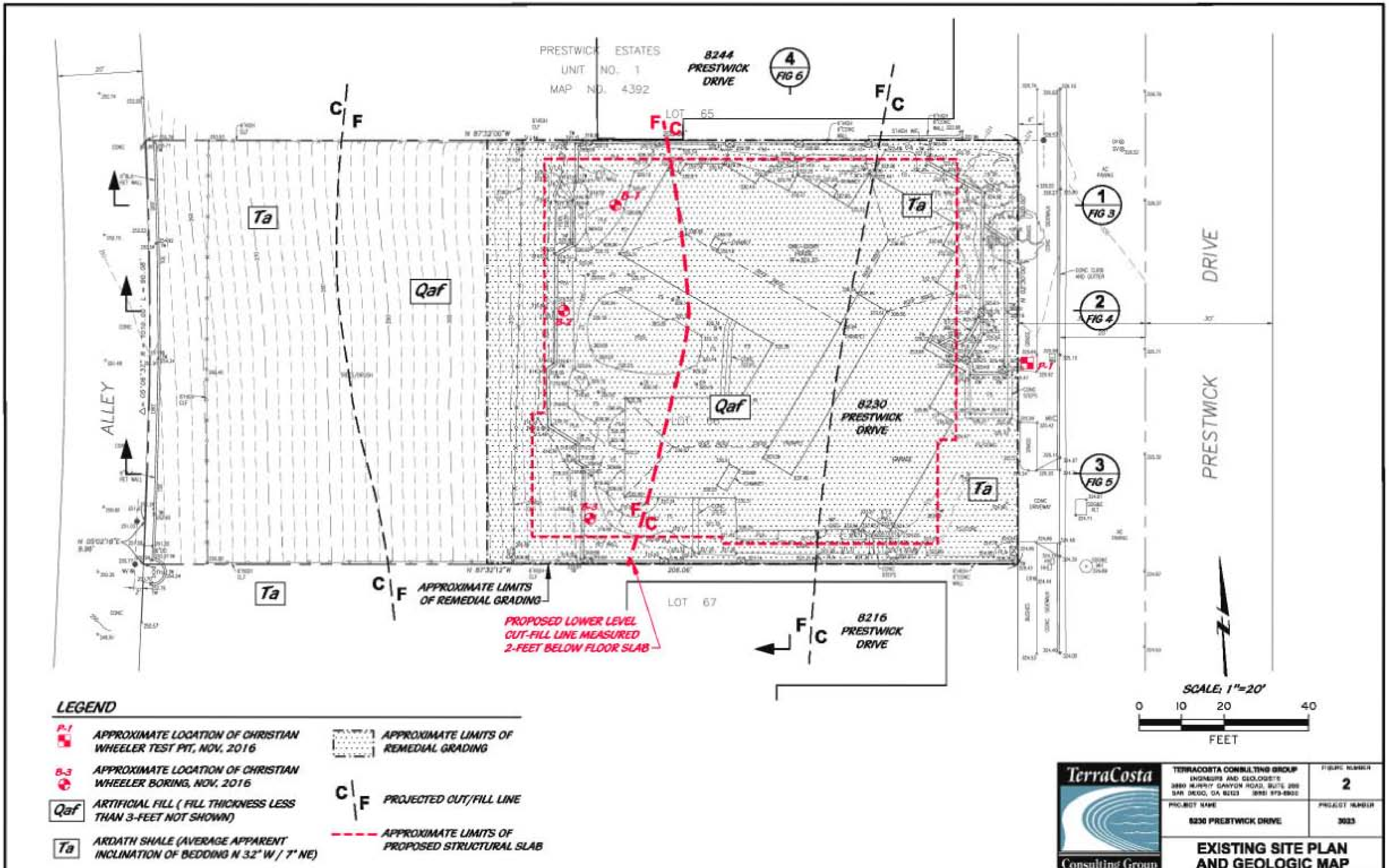
Ta Ardath Shale (middle Eocene) – Mostly uniform, weakly fissile olive-gray silty shale. The upper part contains thin beds of medium-grained sandstone, similar to thicker ones in the overlying Scripps Formation, and concretionary beds with molluscan fossils. The type section of the Ardath Shale is on the east side of Rose Canyon, 800 m south of the Ardath Road intersection with Interstate 5 (Kennedy and Moore, 1971).

SOURCE:

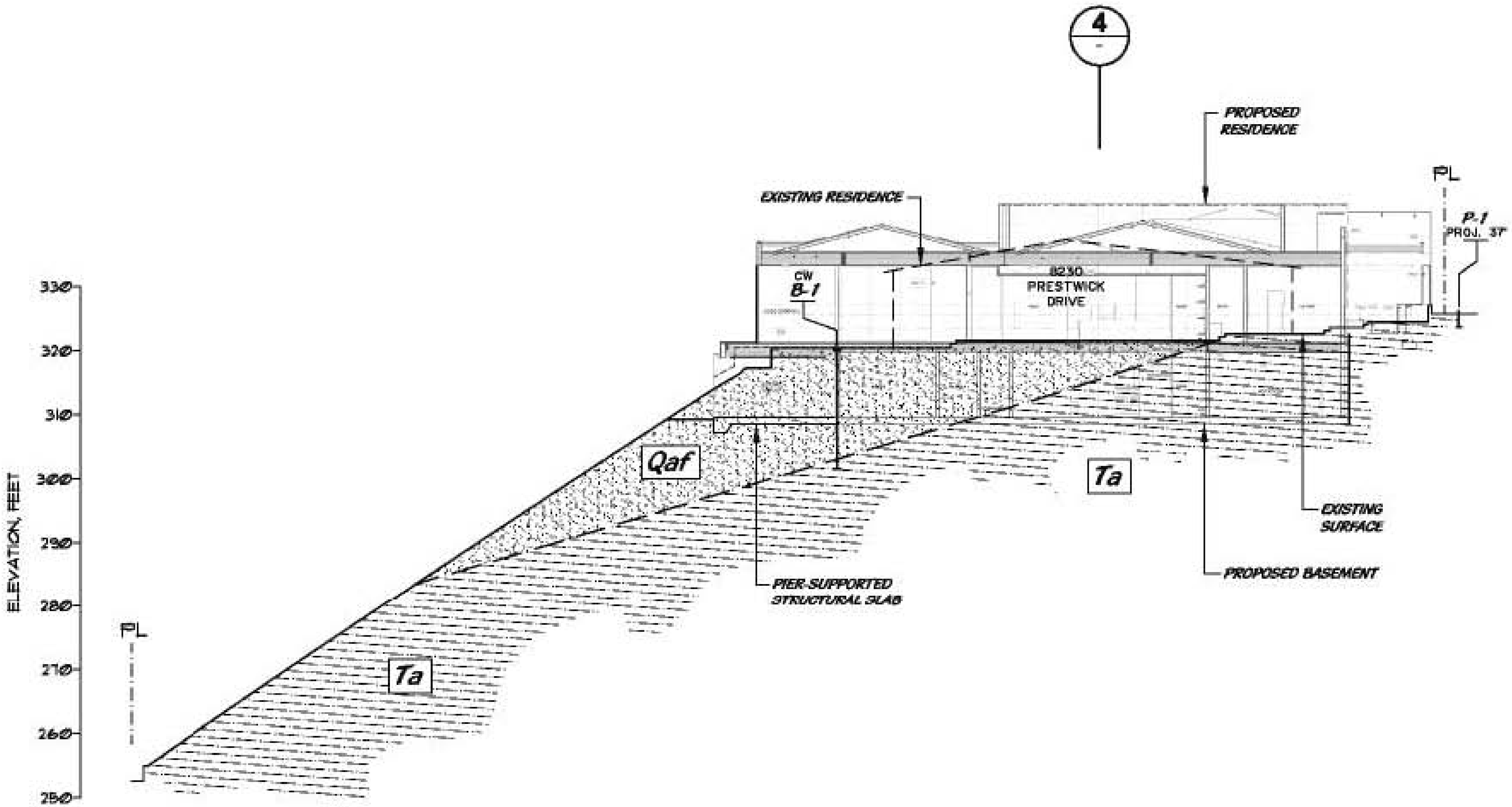
Reproduced from: "Geologic Map of the San Diego 30'x60' Quadrangle, California," by Michael P. Kennedy and Siang S. Tan, 2008.



REGIONAL GEOLOGY MAP



PRESTWICK ESTATES
UNIT NO. 1
MAP NO. 4392
8244 PRESTWICK DRIVE
4 FIG 6
LOT 65
LOT 67
8230 PRESTWICK DRIVE
8216 PRESTWICK DRIVE
1 FIG 3
2 FIG 4
3 FIG 5
PRESTWICK DRIVE
ALLEY
Ta
Qaf
B-1
B-2
B-3
P-1
C/F
F/C
APPROXIMATE LIMITS OF REMEDIAL GRADING
PROPOSED LOWER LEVEL CUT-FILL LINE MEASURED 2-FEET BELOW FLOOR SLAB
SCALE: 1"=20'
0 10 20 40
FEET
LEGEND
P-1 APPROXIMATE LOCATION OF CHRISTIAN WHEELER TEST PIT, NOV. 2016
B-3 APPROXIMATE LOCATION OF CHRISTIAN WHEELER BORING, NOV. 2016
Qaf ARTIFICIAL FILL (FILL THICKNESS LESS THAN 3-FEET NOT SHOWN)
Ta ARDATH SHALE (AVERAGE APPARENT INCLINATION OF BEDDING N 32° W / 7° NE)
APPROXIMATE LIMITS OF REMEDIAL GRADING
C/F PROJECTED CUT/FILL LINE
APPROXIMATE LIMITS OF PROPOSED STRUCTURAL SLAB
TerraCosta Consulting Group
TERRACOSTA CONSULTING GROUP
ENGINEERS AND GEOLOGISTS
FIGURE NUMBER 2
PROJECT NAME 8230 PRESTWICK DRIVE
PROJECT NUMBER
EXISTING SITE PLAN AND GEOLOGIC MAP



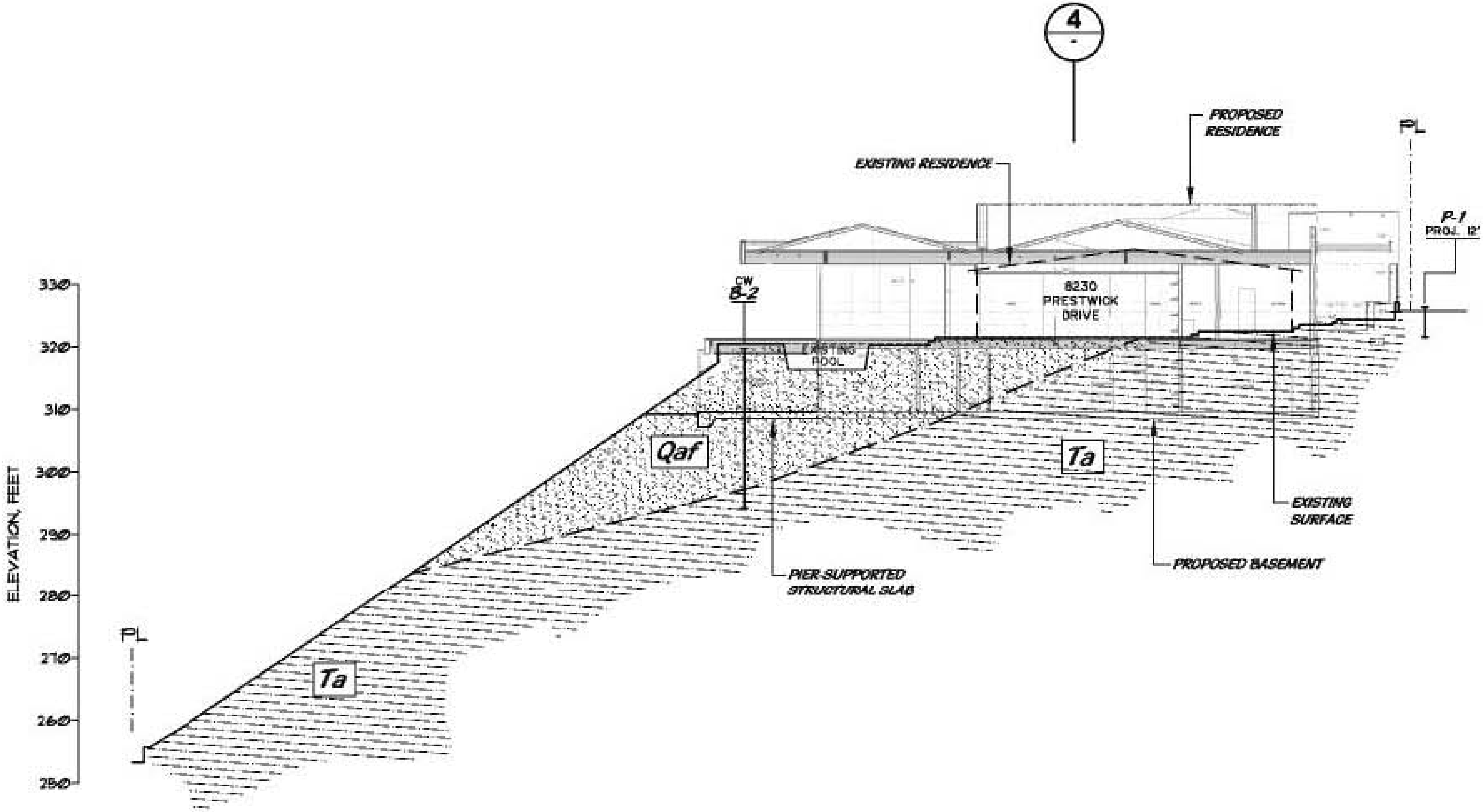
CROSS SECTION 1

SCALE: 1"=20'

LEGEND

CW B-1	APPROX. LOCATION OF CHRISTIAN-WHEELER BORING
— — —	GEOLOGIC CONTACT
Qaf	FILL
Ta	ARDATH FORMATION

TerraCosta Consulting Group	TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER 3
	PROJECT NAME 8230 PRESTWICK DRIVE, LA JOLLA	PROJECT NUMBER 3023
	CROSS SECTION	



CROSS SECTION 2

SCALE: 1"=20'

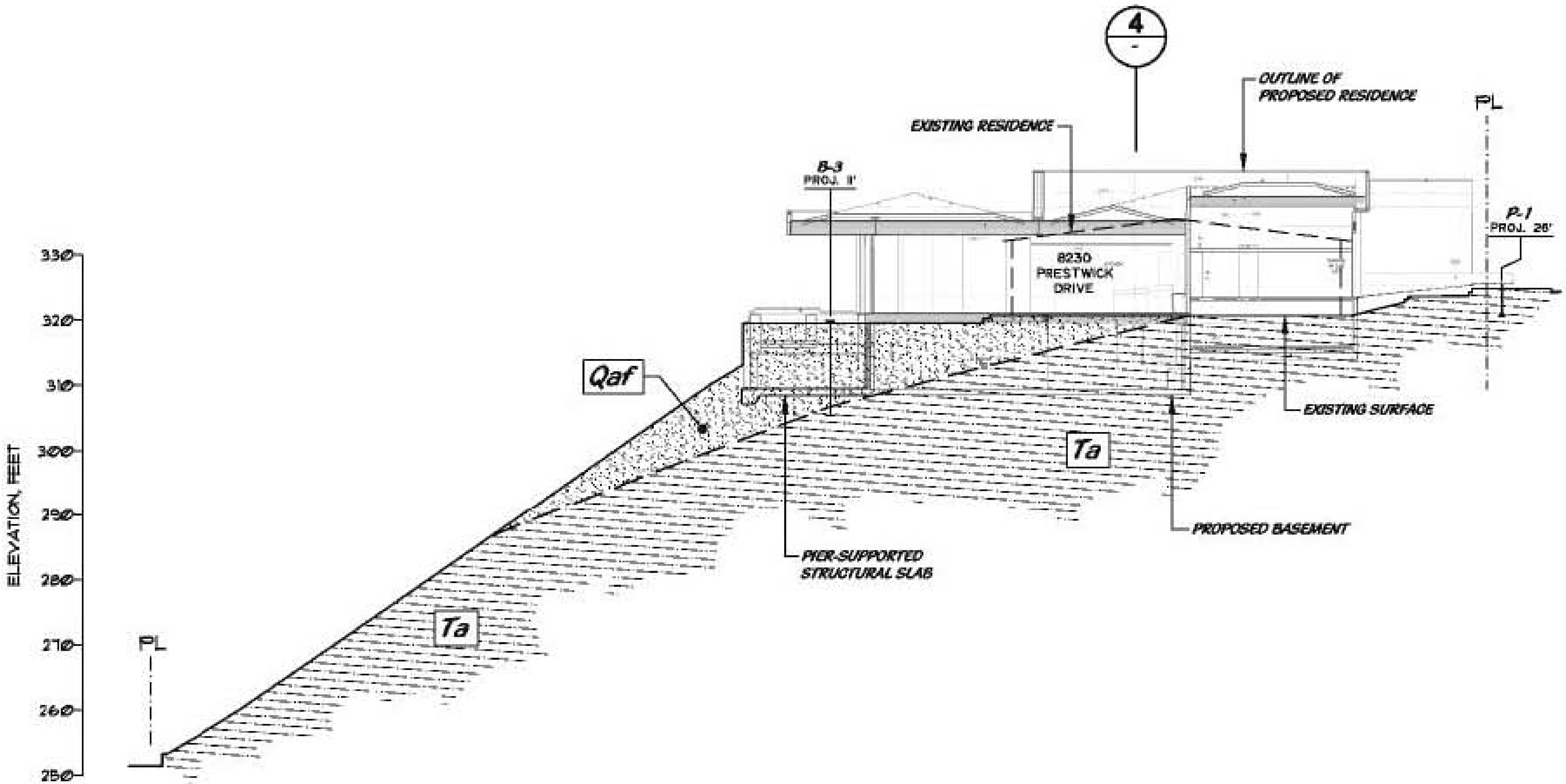
LEGEND

CW B-2	APPROX. LOCATION OF CHRISTIAN-WHEELER BORING
— — —	GEOLOGIC CONTACT
Qaf	FILL
Ta	ARDATH FORMATION

TerraCosta Consulting Group

TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER 4
PROJECT NAME 8230 PRESTWICK DRIVE, LA JOLLA	PROJECT NUMBER 3023

CROSS SECTION



CROSS SECTION 3
-

SCALE: 1"=20'

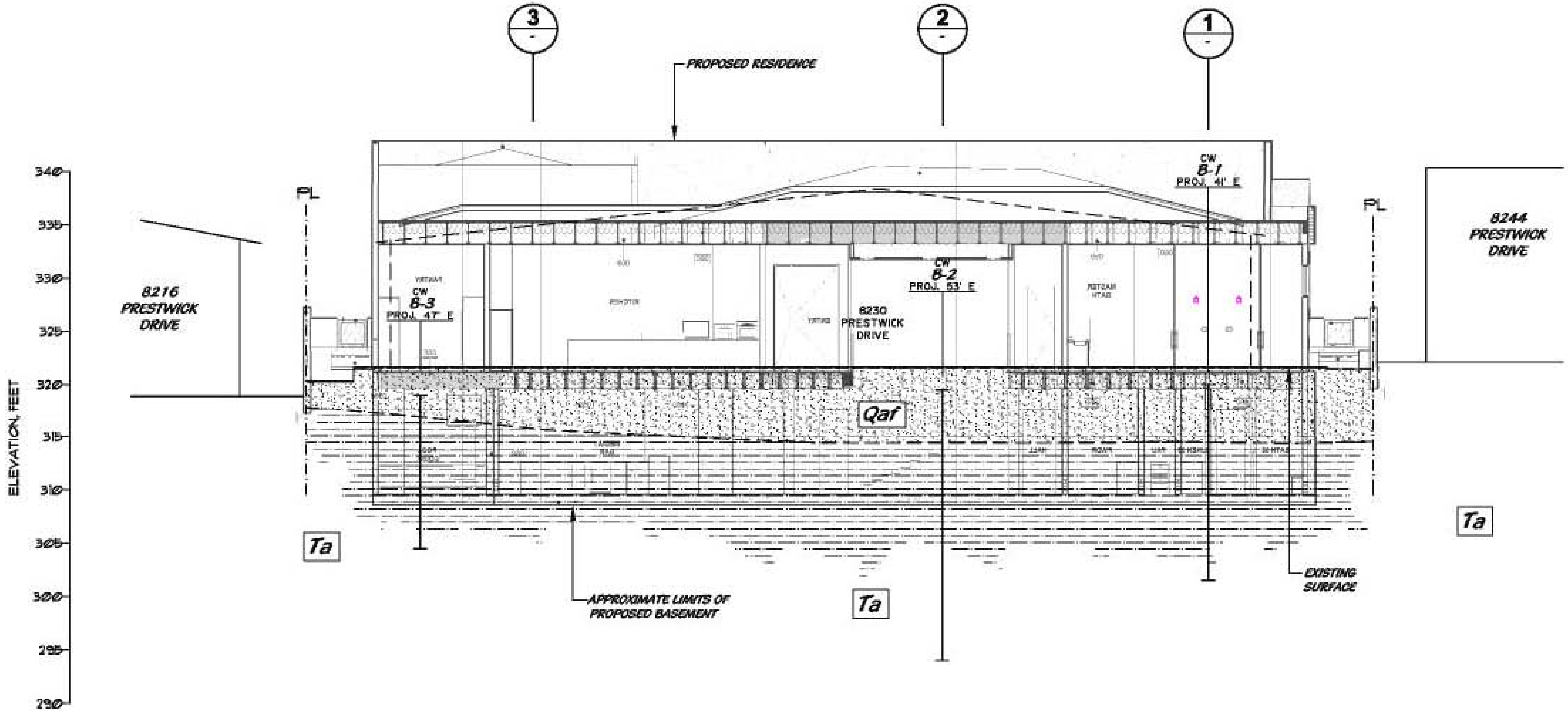
LEGEND

CW B-3	APPROX. LOCATION OF CHRISTIAN-WHEELER BORING
— — —	GEOLOGIC CONTACT
Qaf	FILL
Ta	ARDATH FORMATION

TerraCosta Consulting Group

TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3030 MURPHY CANYON ROAD, SUITE 205 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER 5
PROJECT NAME 8230 PRESTWICK DRIVE, LA JOLLA	PROJECT NUMBER 3023

CROSS SECTION



CROSS SECTION 4

SCALE: 1"=20'

LEGEND

CW B-3 — APPROX. LOCATION OF CHRISTIAN-WHEELER BORING

— — — GEOLOGIC CONTACT

Qaf — FILL

Ta — ARDATH FORMATION

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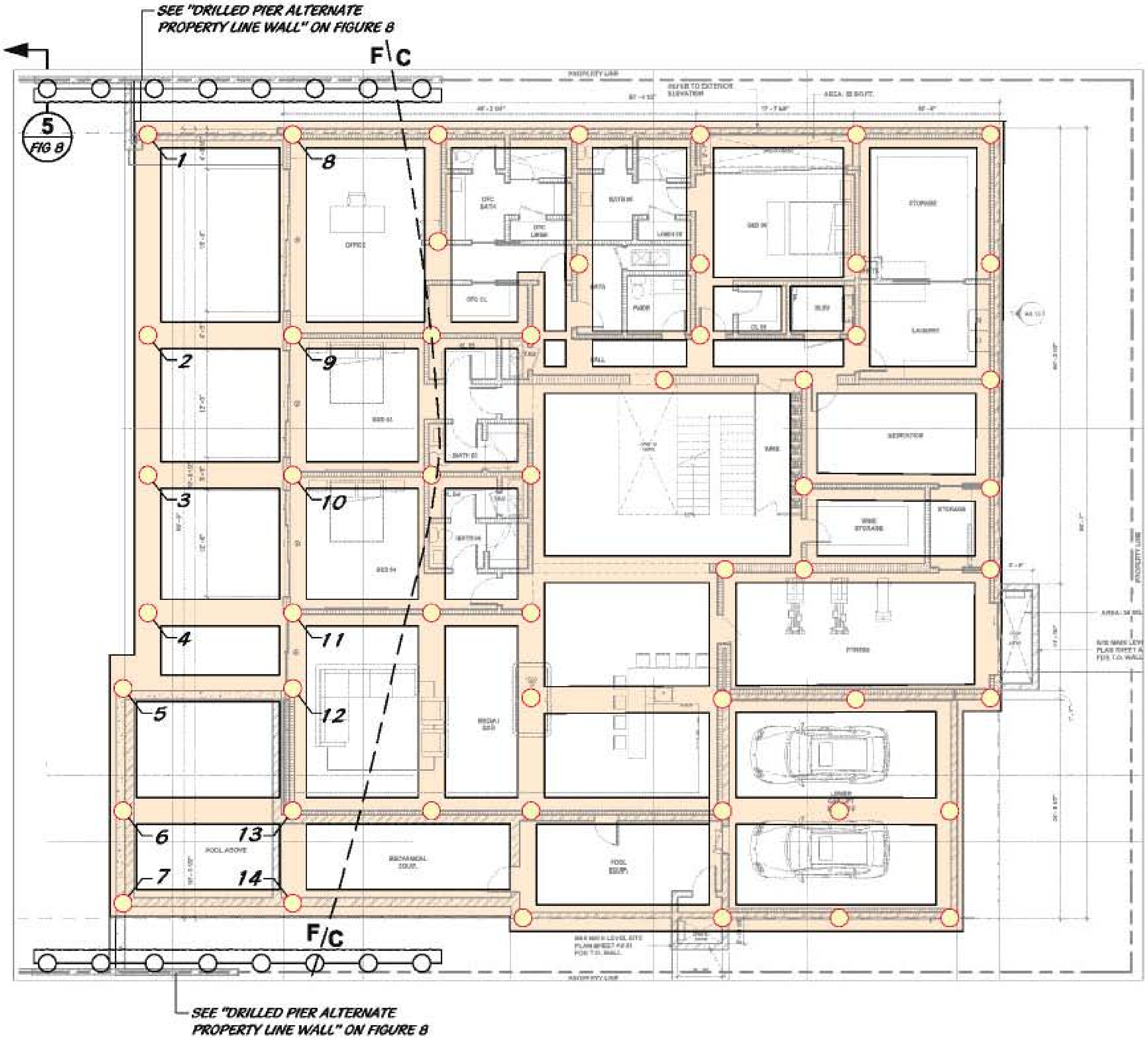
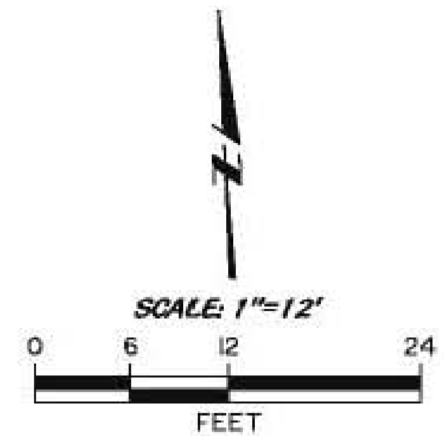
TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER 6
PROJECT NAME 8230 PRESTWICK DRIVE	PROJECT NUMBER 3023

CROSS SECTION





LEGEND

1	PROPOSED 2' DIAMETER DRILLED PIER NO. 1
	PROPOSED 3' WIDE CONCRETE GRADE BEAM
	PROPOSED 12" WIDE CONCRETE GRADE BEAM
C/F	PROPOSED LOWER LEVEL CUT-FILL LINE MEASURED 2-FEET BELOW FLOOR SLAB

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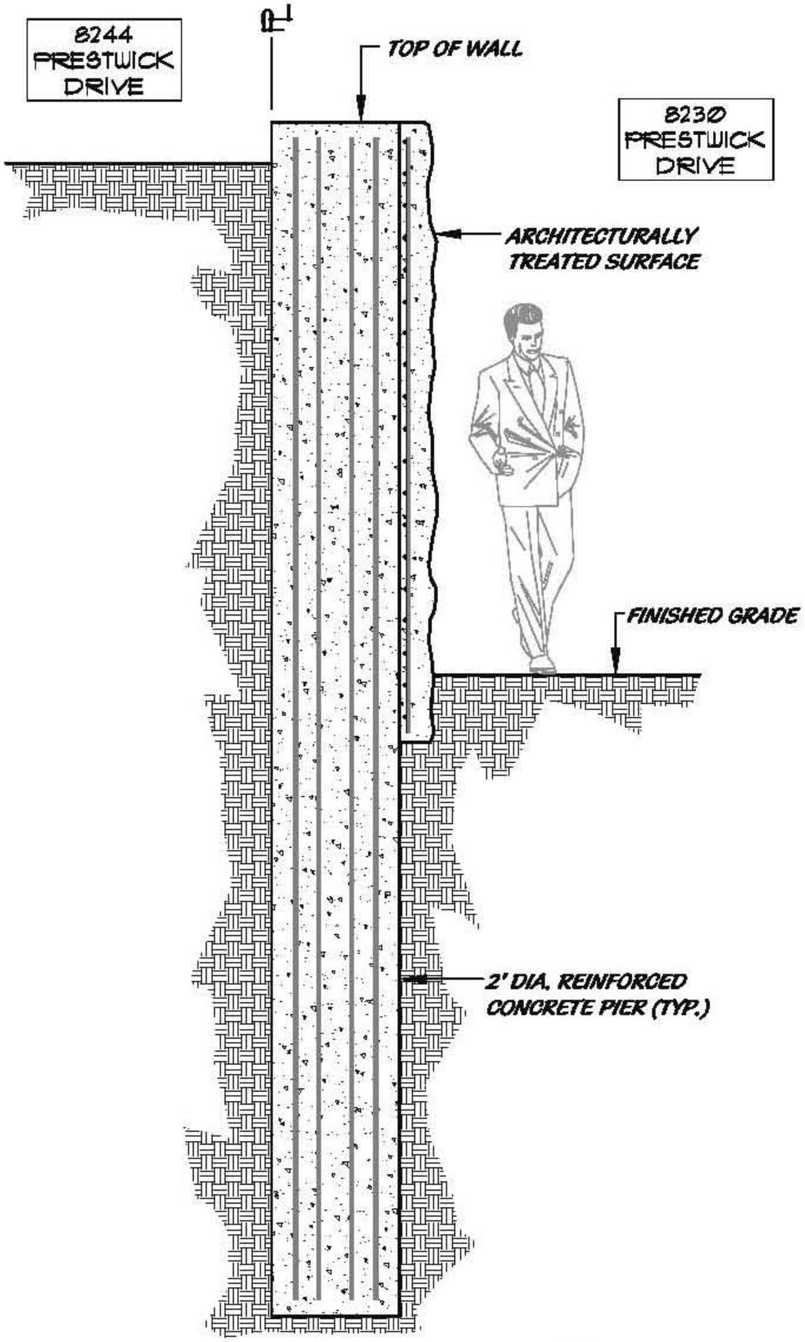
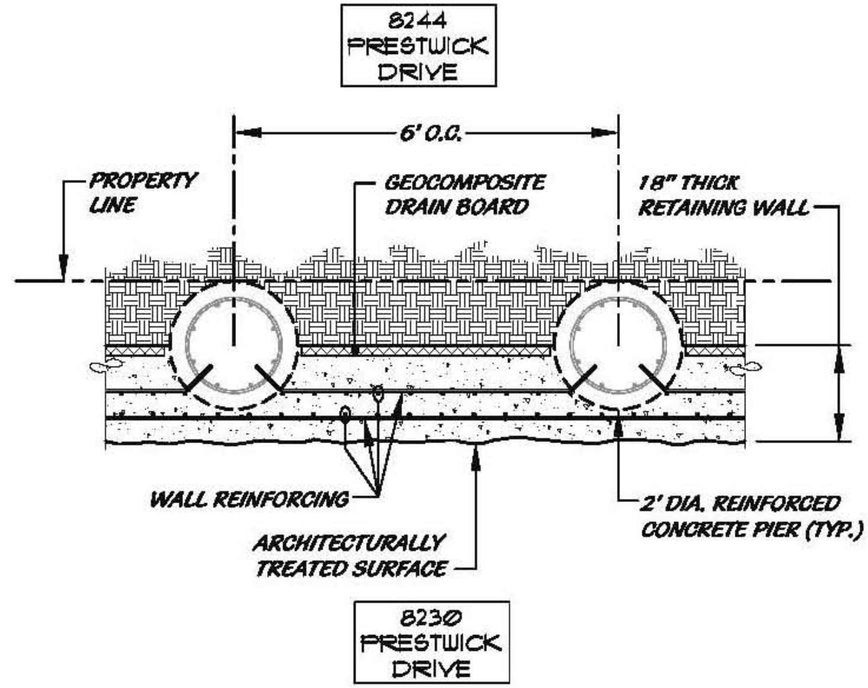
TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER 7
PROJECT NAME 8230 PRESTWICK DRIVE	PROJECT NUMBER 3023

PROPOSED LOWER LEVEL FOUNDATION PLAN



CROSS SECTION 5
-

SCALE: 1"=3' (HORIZ. & VERT.)



PROPERTY LINE WALL
NORTH WALL SHOWN

SCALE: 1"=3'

TerraCosta Consulting Group	TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER 8
	PROJECT NAME 8230 PRESTWICK DRIVE	PROJECT NUMBER 3023
	DRILLED PIER ALTERNATE PROPERTY LINE WALL	

APPENDIX A

REPORT OF PRELIMINARY
GEOTECHNICAL INVESTIGATION
BY CHRISTIAN WHEELER ENGINEERING
DATED NOVEMBER 30, 2016

CHRISTIAN WHEELER
ENGINEERING

REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED HERSHFIELD RESIDENCE
8230 PRESTWICK DRIVE
LA JOLLA, CALIFORNIA

PREPARED FOR

LARRY HERSHFILED
POST OFFICE BOX 7202
RANCHO SANTE FE, CALIFORNIA 92077

PREPARED BY

CHRISTIAN WHEELER ENGINEERING
3980 HOME AVENUE
SAN DIEGO, CALIFORNIA 92105

CHRISTIAN WHEELER
ENGINEERING

November 30, 2016

Larry Hershfield
Post Office Box 7202
Rancho Santa Fe, California 92077

CWE 2160443.02

Subject: **Report of Preliminary Geotechnical Investigation**
Proposed Hershfield Residence, 8230 Prestwick Drive, La Jolla, California

Dear Mr. and Mrs. Hershfield:

In accordance with your request and our proposal dated July 22, 2016, we have completed a geotechnical investigation for the subject project. We are presenting herewith a report of our findings and recommendations.

It is our professional opinion and judgment that no geotechnical conditions exist on the subject property that would preclude the construction of the proposed residential structure provided the recommendations presented herein are followed.

If you have questions after reviewing this report, please do not hesitate to contact our office. This opportunity to be of professional service is sincerely appreciated.

Respectfully submitted,
CHRISTIAN WHEELER ENGINEERING

PROFESSIONAL GEOLOGIST
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PROPOSED HERSHFIELD RESIDENCE

8230 PRESTWICK DRIVE

LA JOLLA, CALIFORNIA

INTRODUCTION AND PROJECT DESCRIPTION

This report presents the results of a preliminary geotechnical investigation performed for the proposed residential structure to be located at 8230 Prestwick Drive, La Jolla, California. The following Figure No. 1 presents a vicinity map showing the location of the property.

We understand that the existing structure and associated improvements on-site will be demolished. A new one- and/or two-story residential structure that will include a basement is proposed. We anticipate that the above-grade portion of the proposed structure will be of conventional, wood-frame construction whereas the basement of the residence will be of concrete/masonry construction. An infinity edge swimming pool is also proposed. We also anticipate that the proposed construction will be supported by drilled cast-in-place concrete piers and conventional shallow foundations. Grading to accommodate the proposed improvements is expected to consist of cuts of up to about 10 feet from existing site grades to accommodate the basement level of the proposed residence.

To assist in the preparation of this report, we have obtained several geotechnical reports prepared by Geotechnical Exploration, Inc. and others prepared before and after the construction of the existing structure. In addition, we have reviewed miscellaneous drawings prepared by James D. Dodge Architect, of unknown date.

A Google Earth image was used as a base map for our Site Plan and Geologic Map and geologic cross sections. The Site Plan and Geologic Map and geologic cross sections are included herein as Plate Nos. 1 through 3.

SITE VICINITY

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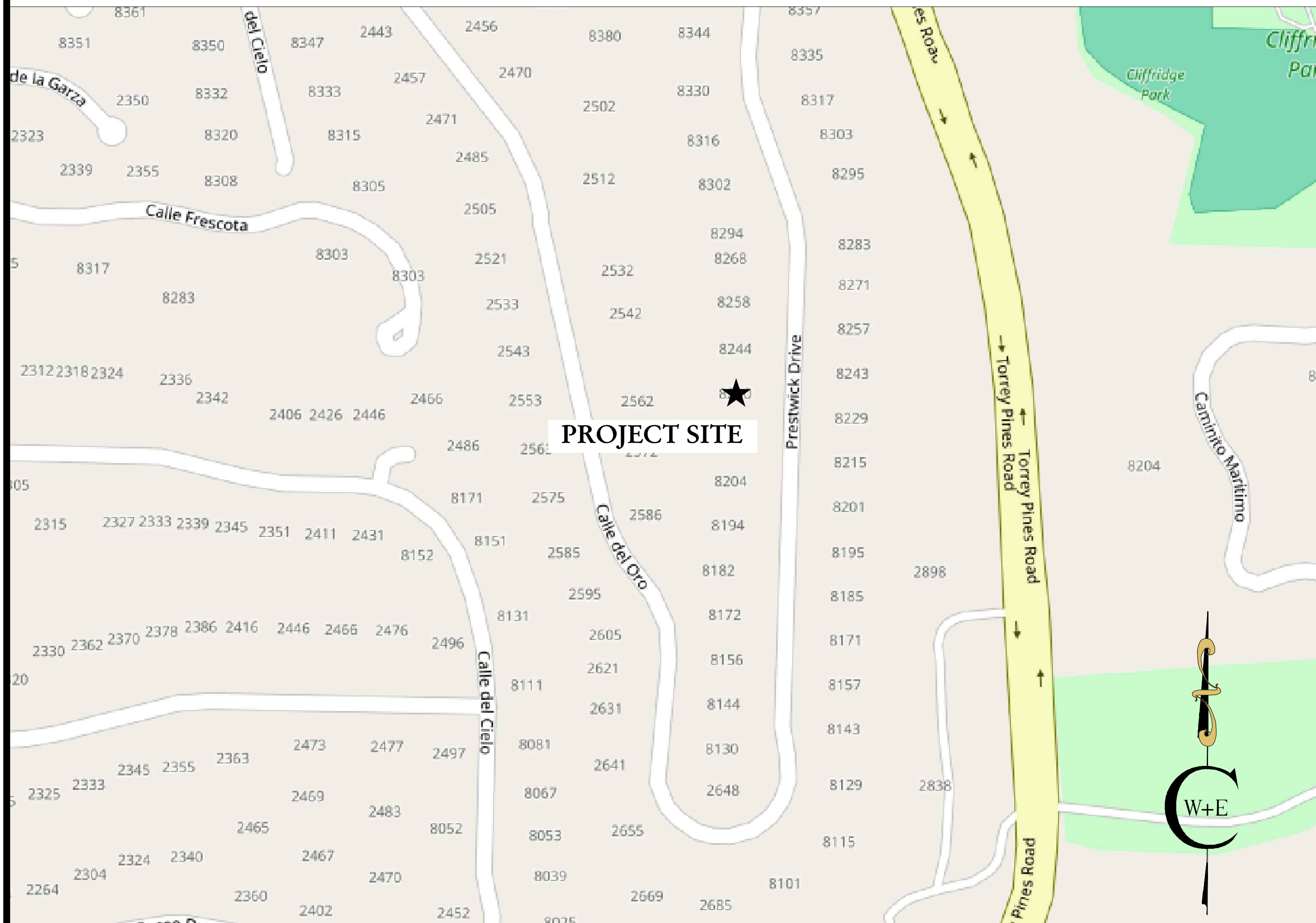


HERSHFIELD RESIDENCE
8230 PRESTWICK DRIVE
SAN DIEGO, CALIFORNIA

DATE:	NOVEMBER 2016	JOB NO.:	2160443.02
BY:	SRD	FIGURE NO.:	1

CHRISTIAN WHEELER
ENGINEERING

This report has been prepared for the exclusive use of Larry Hershfield, and his design consultants, for specific application to the project described herein. Should the project be modified, the conclusions and recommendations presented in this report should be reviewed by Christian Wheeler Engineering for conformance with our recommendations and to determine whether any additional subsurface investigation, laboratory testing and/or recommendations are necessary. Our professional services have been performed, our findings obtained and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, expressed or implied.

SCOPE OF SERVICES

Our preliminary geotechnical investigation consisted of surface reconnaissance, subsurface exploration, obtaining representative soil samples, laboratory testing, analysis of the field and laboratory data, and review of relevant geologic literature. Our scope of service did not include assessment of hazardous substance contamination, recommendations to prevent floor slab moisture intrusion or the formation of mold within the structures, evaluation or design of storm water infiltration facilities, or any other services not specifically described in the scope of services presented below.

More specifically, the intent of our proposed investigation was to:

- Drill three small diameter borings at the site with a limited access, tripod mounted drill rig and excavate a hand-dug test pit to explore the existing soil conditions.
- Backfill the boring holes using a grout or a grout/bentonite mix as required by the County of San Diego Department of Environmental Health.
- Research the our files and the city of San Diego files for pertinent information regarding the as-built conditions at the site and geotechnical reports for the general site vicinity.
- Evaluate, by laboratory tests and our past experience with similar soil types, the engineering properties of the various soil strata that may influence the proposed construction, including bearing capacities, expansive characteristics and settlement potential.
- Describe the general geology at the site, including possible geologic hazards that could have an effect on the proposed construction, and provide the seismic design parameters as required by the 2013 edition of the California Building Code.

- Address potential construction difficulties that may be encountered due to soil conditions, groundwater or geologic hazards, and provide geotechnical recommendations to deal with these difficulties.
- Provide site preparation and grading recommendations for the anticipated work.
- Provide foundation recommendations for the type of construction anticipated and develop soil engineering design criteria for the recommended foundation designs.
- Provide design parameters for restrained and unrestrained retaining walls.
- Provide a preliminary geotechnical report that presents the results of our investigation which includes a plot plan showing the location of our subsurface explorations, excavation logs, laboratory test results, and our conclusions and recommendations for the proposed project.

Although a test for the presence of soluble sulfates within the soils that may be in contact with reinforced concrete was performed as part of the scope of our services, it should be understood Christian Wheeler Engineering does not practice corrosion engineering. If a corrosivity analysis is considered necessary, we recommend that the client retain an engineering firm that specializes in this field to consult with them on this matter. The results of our sulfate testing should only be used as a guideline to determine if additional testing and analysis is necessary.

FINDINGS

SITE DESCRIPTION

The subject site is a rectangular-shaped, developed residential lot located at 8230 Prestwick Drive, La Jolla, California. The lot is also bounded to the east by Prestwick Drive, on the north and south by developed, residential lots and to the west by a paved alleyway. The site currently supports a single-story residential structure, a swimming pool, and typical exterior improvements. It is our understanding that the existing structure is supported by a combination of shallow foundations and drilled cast-in-place concrete piers. A total of thirteen piers are located at the northwestern corner of the building and are designed to extend at least 3 feet into the Ardath Shale. Topographically, the developed portion of the site is relatively level. However, a descending fill over natural slope approximately 75 feet high exists at the western portion of the site. The slope has an estimated inclination of about 1.5:1 (horizontal to vertical) or flatter. A retaining wall about 4 feet in height exists at the top of the slope.

GENERAL GEOLOGY AND SUBSURFACE CONDITIONS

GEOLOGIC SETTING AND SOIL DESCRIPTION: The subject site is located within the Coastal Plains Physiographic Province of San Diego County. Based on the results of our subsurface explorations, and analysis of readily available, pertinent geologic literature, it was determined that the site is generally underlain by artificial fill and Ardath Shale (see Plate Nos. 1, 2 and 3). These materials are described below:

> **ARTIFICIAL FILL (Qaf):** Artificial fill was encountered underlying the site. As encountered in the subsurface explorations, the artificial fill comprises a wedge that increases in depth to the west. The artificial fill extends to a depth of about 17½ feet, 24 feet, and 13 feet below existing grade in borings B-1, B-2, and B-3, respectively. The fill soils in test pit P-1 extend to a depth of about 1 foot below existing grade. The fill materials generally consisted of brown, olive brown, and yellowish-brown, moist and very moist, soft to stiff, sandy silty clay (CL). The upper 2½ feet of fill soils in boring B-1 consisted of brown to grayish-brown, moisty and very moist, loose and medium dense, silty sand. The clayey artificial fill was found to have a low expansion potential (EI=46), whereas the sandy artificial fill was judged to have a very low expansion potential (EI<20).

> **ARDATH SHALE (Ta):** Tertiary-age sedimentary Ardath Shale was encountered underlying the artificial fill. These deposits generally consisted of light gray to yellowish-brown, moist, hard, clayey silt-silty clay (ML-CL) and silty clay (CL). The Ardath shale was judged to have a medium expansion potential (EI between 51 and 90).

GEOLOGIC STRUCTURE: Based on our review of the referenced geologic maps and our experience in the vicinity of the subject site, the bedding of the Tertiary-age sedimentary deposits that underlie the site dips gently (<5°) to the northeast. Such bedding orientation is generally considered to be favorable with regards to the stability of the westerly sloping site.

GROUNDWATER: Minor seepage was encountered in boring B-3 at the contact between the artificial fill and the Ardath Shale; however, similar groundwater conditions were not observed in the other two borings or test pit. We do not anticipate any significant groundwater related conditions during or after the proposed construction. However, it should be recognized that minor groundwater seepage problems might occur after construction and landscaping are completed, even at a site where none were present before construction. These are usually minor phenomena and are often the result of an alteration in drainage patterns and/or an increase in irrigation water. Based on the anticipated construction and the permeability of the on-site soils, it is our opinion that any seepage problems that may occur will be minor in extent. It is further our opinion that these problems can be most effectively corrected on an individual basis if and when they occur.

TECTONIC SETTING: It should be noted that much of Southern California, including the San Diego County area, is characterized by a series of Quaternary-age fault zones that consist of several individual, en echelon faults that generally strike in a northerly to northwesterly direction. Some of these fault zones (and the individual faults within the zone) are classified as active while others are classified as only potentially active according to the criteria of the California Division of Mines and Geology. Active fault zones are those which have shown conclusive evidence of faulting during the Holocene Epoch (the most recent 11,000 years) while potentially active fault zones have demonstrated movement during the Pleistocene Epoch (11,000 to 1.6 million years before the present) but no movement during Holocene time. Inactive faults are those faults that can be demonstrated to have no movement in the past 1.6 million years.

It should be recognized that the active Rose Canyon Fault Zone is located approximately ½ mile southwest of the site. Other active fault zones in the region that could possibly affect the site include the Coronado Bank, San Diego Trough, and San Clemente Fault Zones to the west, the Palos Verdes and Newport Inglewood Fault Zones to the northwest, and the Elsinore, Earthquake Valley, San Jacinto and San Andreas Fault Zones to the northeast.

GENERAL GEOLOGIC HAZARDS

GENERAL: The site is located in an area where the risks due to significant geologic hazards are relatively low. No geologic hazards of sufficient magnitude to preclude the construction of the subject project are known to exist. In our professional opinion and to the best of our knowledge, the site is suitable for the proposed improvements.

CITY OF SAN DIEGO SEISMIC SAFETY STUDY: As part of our services, we have reviewed the City of San Diego Seismic Safety Study. This study is the result of a comprehensive investigation of the City that rates areas according to geological risk potential (nominal, low, moderate, and high) and identifies potential geotechnical hazards and/or describes geomorphic conditions. According to the San Diego Seismic Safety Map No. 29, the site is located in Geologic Hazard Category 26. Hazard Category 26 is assigned to areas underlain by slide prone formations, specifically Ardath Shale, with unfavorable geologic structure, where the relative level of geologic risk is considered to be "moderate."

SURFACE RUPTURE: There are no known active faults that traverse the subject site; therefore, the risk for surface rupture at the subject site is considered low.

LIQUEFACTION: The earth materials underlying the site are not considered subject to liquefaction due to such factors as soil density, grain-size distribution, the absence of shallow groundwater conditions.

FLOODING: As delineated on the Flood Insurance Rate Map (FIRM) prepared by the Federal Emergency Management Agency, the site is not located within either the 100-year flood zone or the 500-year flood zone.

TSUNAMIS: Tsunamis are great sea waves produced by submarine earthquakes or volcanic eruptions. Due to the site's setback from the ocean and elevation, it will not be affected by a tsunami.

SEICHES: Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays or reservoirs. Due to the site's location, it will not be affected by seiches.

SLOPE STABILITY: As part of this investigation we reviewed the publication, "Landslide Hazards in the Southern Part of the San Diego Metropolitan Area" by Tan, 1995. This reference is a comprehensive study that classifies San Diego County into areas of relative landslide susceptibility. The subject site is located in Area 4-1, which is considered to be "most susceptible" to slope failures.

Based on our findings and the proposed construction, it is our opinion that the likelihood of deep seated slope stability related problems at the site is low. The following presents descriptions of our global and surficial stability analyses.

SLOPE STABILITY ANALYSES

GENERAL: In consideration of the sloping topography at the rear of the subject site, we have performed a quantitative, global stability analysis to determine the site's minimum factor-of-safety against deep-seated slope failure. It is our professional opinion that the cross section modeled in our stability analyses, oriented perpendicular to the slope, conservatively models the proposed site configuration. We have also performed a surficial stability analysis to determine the minimum factor-of-safety against surficial failure. Descriptions of our stability analyses are presented in the following "Gross Stability Analyses" and "Surficial Stability Analyses" sections of this report.

GROSS STABILITY ANALYSES

CROSS-SECTIONS: As presented on our Site Plan and Geotechnical Map, included herein as Plate No. 1, we have created two geologic cross sections to depict the proposed topography and subsurface conditions at the subject site. The geologic cross sections are included on Plate No. 2 and 3 of this report. The locations of the geologic cross section were chosen to be oriented perpendicular to the topography of the slope and included the steepest portions of the sloping site.

To analyze the stability of the subject site we have performed a series of quantitative slope stability analyses incorporating the topography and geologic conditions presented on our geologic cross section B-B', which represents the worst case scenario on the site. The on-site earth materials incorporated in our stability analyses are described above in the "Geologic Setting and Soil Description" section of this report. Based on the composition of the underlying formational material and the geologic structure of the area circular- type failure mechanisms were modeled in our analyses. The results of our quantitative slope stability analyses are presented below in the results of Stability Analyses Section of this report.

STRENGTH PARAMETERS: The strength parameters for the earth materials underlying the subject site were estimated by the direct shear test method and our experience and judgment with similar soil types. The results of our direct shear testing are presented at the rear of this report. The unit weights of the earth materials that underlie the subject site and adjacent areas utilized in our stability analyses were chosen based on the results of our laboratory testing and our experience with similar materials in the vicinity of the subject site. It is our professional opinion that the strength parameters and unit weights presented below and utilized in our stability analyses provide for conservative slope stability analyses.

Soil Type	Unit Weight, γ	Phi, ϕ	Cohesion, c
Artificial Fill	120 pcf	26°	400 psf
Ardath Shale	125 pcf	28°	650 psf

METHOD OF ANALYSES: The analyses of the gross stability of the proposed site topography were performed using Version 2 of the GSTABL7© computer program developed by Garry H. Gregory, PE. The program analyzes circular, block, specified, and randomly shaped failure surfaces using the Modified Bishop, Janbu, or Spencer's Methods. The STEDwin© computer program, developed by Harald W. Van Aller, P. E., was used in conjunction with this program for data entry and graphics display. The proposed topography of the subject site along geologic cross section B-B' were analyzed for circular failures and each failure analysis was programmed to run at least 2,000 random failure surfaces. The most critical failure surfaces were then accumulated and sorted by value of the factor-of-safety. After the specified number of failure surfaces were successfully generated and analyzed, the ten most critical surfaces were plotted so that the pattern may be studied.

RESULTS OF STABILITY ANALYSES: Appendix E of this report presents the results of our static and pseudo-static (incorporating a kh value of 0.15g), gross stability analyses. As demonstrated on the printouts of these analyses (see Appendix A), the proposed site topography along our geologic cross section B-B' demonstrates minimum factors-of-safety greater than 1.5 and 1.1 against static and pseudo-static failures, respectively, which are the minimums that are generally considered to be stable.

SURFICIAL SLOPE STABILITY

GENERAL: Appendix E of this report presents the results of our surficial slope stability analysis of those portions of the existing fill slope along the west side of the lot. As demonstrated on the printout of this analysis, the existing fill slope demonstrates a minimum factor-of-safety against surficial slope failure of 1.8 where the saturation depth is 5 feet, which is higher than the minimum that are generally considered to be stable at 1.5. However, care should be taken by the project contractor and homeowner to minimize the amount of water allowed on the slope.

CONCLUSIONS

In general, it is our professional opinion and judgment that the subject property is suitable for the construction of the proposed structure. The main geotechnical conditions affecting the proposed project consist of artificial fill and the existing slope at the west portion of the property. Other geotechnical conditions and issues that will affect the proposed construction are a cut/fill transition, expansive soils, minor seepage encountered in one of our borings, temporary cut slopes, and existing pier foundations. These conditions are discussed hereinafter.

The site is underlain by a wedge of artificial fill. As encountered in subsurface explorations, it appears that within the eastern portion of the property the fill is relatively shallow (see Plate Nos.1, 2 and 3). Within the rest of the site, the artificial fill increases in depth to the west to a maximum of about 24 feet below existing grade (boring B-2). Deeper fill soils may exist in areas of the site not investigated. The fill soils appear to be potentially compressible. A full basement is anticipated under the proposed structure. The proposed basement construction will remove a large portion of the fill soils under the proposed structure, and will result in the presence of fill soils and Ardath Shale at proposed finish pad grade. This configuration may result in differential settlement detrimental to the proposed structure. In order to mitigate this condition, the foundation system for the proposed structure should extend through the fill into the underlying Ardath Shale. This recommendation will result on drilled cast-in-place concrete piers being necessary for the support of at least the western portion of the structure. In addition, piers will be needed for the support of the proposed pool. Furthermore, partial removal and replacement of existing fill under proposed exterior improvements will be necessary.

The prevailing foundation soils were judged to be moderately expansive (EI between 51 and 90). The foundation recommendations contained hereinafter to mitigate other geotechnical conditions also mitigate for expansive soils. However, soils with medium expansion potential may detrimentally affect light-weight exterior improvements such as site walls, sidewalks, and driveways. Select grading consisting of replacing the expansive soils with a soil that has a low expansive potential is one of the best ways to mitigate for expansive soil conditions. It is assumed that select grading as recommended hereinafter will be performed as part of the project. If select grading is unfeasible, consideration should be given to utilizing materials that are tolerant to movement, implementing drought tolerant landscaping, providing positive drainage away from exterior improvements, and providing concrete surfaces with appropriate weakened plane joints. Regardless of these or other similar measures, some distress to exterior improvements requiring future maintenance or even replacement should be anticipated due to expansive soils.

Seepage was encountered in boring B-3 at a depth of about 12 feet below existing grade. The seepage occurs at the contact between the fill and Ardath Shale. It appears that this is a localized condition. It is anticipated that this condition will not greatly affect the proposed construction.

Temporary cut slopes up to about 12 feet height will be necessary for the proposed basement construction. It is anticipated that shoring will be necessary along the southern and northern property lines. Recommendations for shored and unshored temporary cut slopes are provided hereinafter.

Thirteen concrete piers are expected to be located at the northwestern corner of the existing structure. It is anticipated that the piers will have to be partially demolished in order to construct the proposed basement. These piers are considered unsuitable to support the new improvements.

The site is located in an area that is relatively free of geologic hazards that will have a significant effect on the proposed construction. The most likely geologic hazard that could affect the site is ground shaking due to seismic activity along one of the regional active faults. However, construction in accordance with the requirements of the most recent edition of the California Building Code and the local governmental agencies should provide a level of life-safety suitable for the type of development proposed.

RECOMMENDATIONS

GRADING AND EARTHWORK

GENERAL: All grading should conform to the guidelines presented in the current edition of the California Building Code, the minimum requirements of the City of San Diego, and the recommended Grading Specifications and Special Provisions attached hereto, except where specifically superseded in the text of this report.

PREGRADE MEETING: It is recommended that a pregrade meeting including the grading contractor, the client, and a representative from Christian Wheeler Engineering be performed, to discuss the recommendations of this report and address any issues that may affect grading operations.

OBSERVATION OF GRADING: Continuous observation by the Geotechnical Consultant is essential during the grading operation to confirm conditions anticipated by our investigation, to allow adjustments in design criteria to reflect actual field conditions exposed, and to determine that the grading proceeds in general accordance with the recommendations contained herein.

CLEARING AND GRUBBING: Site preparation should begin with the demolition of existing improvements slated for demolition, and the removal of the resulting debris as well as any existing vegetation and other deleterious materials in areas to receive proposed improvements or new fill soils.

SITE PREPARATION: It is recommended that existing artificial fill underlying proposed exterior light miscellaneous improvements such as hardscape and driveway be removed to a minimum depth of 2 feet below the recommended select cap, whichever is more. Deeper removals may be necessary in areas of the site not investigated or due to unforeseen conditions. Lateral removals limits should extend at least 2 feet from the perimeter of the improvements. No removals are recommended beyond property lines. All excavated areas should be approved by the geotechnical engineer or his representative prior to replacing any of the excavated soils. The excavated materials should be exported from the site and replaced with select imported soils compacted in accordance with the recommendations presented in the "Compaction and Method of Filling" section of this report.

PROCESSING OF FILL AREAS: Prior to placing any new fill soils or constructing any new improvements in areas that have been cleaned out to receive fill, the exposed soils should be scarified to a depth of 12 inches, watered thoroughly, and compacted to at least 90 percent relative compaction.

SELECT FILL SOILS: A minimum of two-foot-thick cap of select imported fill soils is recommended underneath proposed exterior improvements. Select soils are also recommended for retaining wall backfill. Select soils should consist of silty sands and clayey sands that have a low expansive potential (EI between 21 and 50), relatively high shear strength, and relatively low permeability. At least 72 hours advance notice is necessary to properly evaluate the suitability of proposed select imported fill soils.

COMPACTION AND METHOD OF FILLING: In general, all structural fill placed at the site should be compacted to a relative compaction of at least 90 percent of its maximum laboratory dry density as determined by ASTM Laboratory Test D1557. Fills should be placed at or slightly above optimum moisture content, in lifts six to eight inches thick, with each lift compacted by mechanical means. Fills should consist of approved earth material, free of trash or debris, roots, vegetation, or other materials determined to be unsuitable by the Geotechnical Consultant. Fill material should be free of rocks or lumps of soil in excess of 3 inches in maximum dimension.

Utility trench backfill within five feet of the proposed structure and beneath all concrete flatwork or pavements should be compacted to a minimum of 90 percent of its maximum dry density.

TEMPORARY SLOPES: We anticipate that temporary excavation slopes will be required for the construction of the subject project. The excavations required for footing construction are considered as part of the temporary slopes. It is anticipated that the majority of temporary cut slopes will be shored. In general, temporary cuts can be excavated at a continuous inclination of 1:1 or flatter. However, the bottom 4 feet of temporary cut slopes exposing competent Ardath Shale may be constructed vertically. We recommend that our firm be contacted to have an engineering geologist observe the temporary cut slopes during grading to ascertain that no unforeseen adverse conditions exist. If adverse conditions are identified, it may be necessary to flatten the slope inclination. No surcharge loads such as soil or equipment stockpiles, vehicles, etc. should be allowed within a distance from the top of temporary slopes equal to half the slope height.

The contractor is solely responsible for designing and constructing stable, temporary excavations and may need to shore, slope, or bench the sides of trench excavations as required to maintain the stability of the excavation sides where the friable sands are exposed. The contractor's "competent person", as defined in the OSHA Construction Standards for Excavations, 29 CFR, Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety process. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations. Christian Wheeler Engineering should be immediately notified if zones of potential instability, sloughing or raveling develop, and mitigation measures should be implemented prior to continuing work.

SURFACE DRAINAGE: The drainage around the proposed improvements should be designed to collect and direct surface water away from proposed improvements and the top of slopes toward appropriate drainage facilities. Rain gutters with downspouts that discharge runoff away from the structures into controlled drainage devices are recommended.

The ground around the proposed improvements should be graded so that surface water flows rapidly away from the improvements without ponding. In general, we suggest that the ground adjacent to structures be sloped away at a minimum gradient of 2 percent. For densely vegetated areas where runoff can be impaired should have a minimum gradient of 5 percent for the first 5 feet from the structure is suggested. It is essential that new and existing drainage patterns be coordinated to produce proper drainage. Pervious hardscape surfaces are not recommended.

Drainage patterns provided at the time of construction should be maintained throughout the life of the proposed improvements. Site irrigation should be limited to the minimum necessary to sustain landscape growth. Over watering should be avoided. Should excessive irrigation, impaired drainage, or unusually high rainfall occur, zones of wet or saturated soil may develop.

TEMPORARY SHORING

GENERAL: Shoring may be necessary for the proposed construction. It is anticipated that the shoring system will utilize soldier beams with wooden lagging. The following design parameters may be assumed to calculate earth pressures on shoring.

Angle of friction	26°
Apparent cohesion	400 pounds per square foot
Soil unit weight	125 pounds per cubic foot (pcf)

Active pressures can be applied to shoring that is capable of rotating 0.002 radians. At-rest pressures should be applied to a shoring system that is unyielding and not able to rotate. These values do not include surcharge loads. Construction surcharge loads should be evaluated on a case-by-case basis. Vertical and lateral movements of the temporary shoring are expected to be small assuming an adequate lateral support system.

FOUNDATIONS

GENERAL: Based on our findings and engineering judgment, the proposed structure, swimming pool and associated retaining walls may be supported by drilled cast-in-place concrete piers extending through the existing fill soils into the underlying Ardath Shale. Where Ardath Shale is at-grade or within shallow depths, conventional shallow foundations may be used. The piers should be connected by grade beams as recommended by the project structural engineer. Miscellaneous light exterior improvements may be supported by conventional shallow continuous and isolated spread footings. The following recommendations are considered the minimum based on the anticipated soil conditions, and are not intended to be lieu of structural considerations. All foundations should be designed by a qualified engineer.

CONCRETE CAST-IN-PLACE PIERS

MINIMUM PIER DIMENSIONS: Cast-in-place concrete pier foundations to support the proposed structure, swimming pool and associated retaining wall should have a minimum diameter of 24 inches. The piers should extend to a minimum depth of 10 feet below the existing grade and 10 feet into materials of Ardath Shale, whichever is more. At this depth, a bearing capacity of 6,000 pounds per square foot (psf) may be assumed for said piers. This bearing pressure may be increased by 800 psf for each additional foot of depth, and 600 psf for each additional foot of width, up to a maximum bearing pressure of 20,000 psf. This value may be increased by one-third when considering wind and/or seismic loads.

PIER REINFORCING: The reinforcing steel for the piers should be specified by the project structural designer. As a minimum, we recommend that the pier reinforcing extend the full depth of the pier excavation.

LATERAL LOADS: Piers located within 20 feet from the top of the slope should be designed to withstand a lateral load equal to an equivalent fluid pressure of 20 pounds per cubic foot acting on the upper 15 feet of the pier.

LATERAL BEARING CAPACITY: The allowable lateral bearing resistance to lateral loads for the portion of the piers embedded into Ardath Shale may be assumed to be 300 pounds per square foot per foot of depth up to a maximum of 3,000 pounds per square foot. This value may be assumed to act on an area equal to twice the pier diameter.

PIER EXCAVATION OBSERVATION AND CLEANING: The pier excavations should be observed by a member from our staff to determine that the minimum embedment recommend in this report is achieved. Prior to placing the steel reinforcing cages, all loose or disturbed soils at the bottom of the pier excavations should be removed. The cleanout of the pier excavations should be approved by the geotechnical engineer.

EXCAVATION CHARACTERISTICS: It is anticipated that the proposed piers may be excavated utilizing conventional equipment in good working condition. However, cemented soils and concretions should be anticipated within the Ardath Shale.

SHALLOW FOUNDATIONS (PROPOSED STRUCTURE)

DIMENSIONS: Spread footings supporting the proposed structure should be embedded at least 18 inches below lowest adjacent finish pad grade and should extend at least 12 inches into Ardath Shale, whichever is more. Continuous and isolated footings should have a minimum width of 12 inches and 24 inches, respectively. Retaining wall footings should be at least 24 inches wide.

BEARING CAPACITY: Spread footings supporting the proposed light exterior improvements may be designed for an allowable soil bearing pressure of 3,000 pounds per square foot (psf).

This value may be increased by 700 psf for each additional foot of embedment depth and 500 psf for each additional foot of width, up to a maximum of 6,000 psf. This value may be increased by one-third for combinations of temporary loads such as those due to wind or seismic loads. Footings located within 10 feet from the face of slopes should be reviewed by this office.

FOOTING REINFORCING: Reinforcement requirements for foundations should be provided by a structural designer. However, based on the expected soil conditions, we recommend that the minimum reinforcing for continuous footings consist of at least 2 No. 5 bars positioned near the bottom of the footing and 2 No. 5 bars positioned near the top of the footing.

LATERAL LOAD RESISTANCE: Lateral loads against foundations may be resisted by friction between the bottom of the footing and the supporting soil, and by the passive pressure against the footing. The coefficient of friction between concrete and soil may be considered to be 0.25. The passive resistance may be considered to be equal to an equivalent fluid weight of 250 pounds per cubic foot. These values are based on the assumption that the footings are poured tight against undisturbed soil. If a combination of the passive pressure and friction is used, the friction value should be reduced by one-third.

SHALLOW FOUNDATIONS (EXTERIOR IMPROVEMENTS)

DIMENSIONS: Spread footings supporting the proposed light exterior improvements should be embedded at least 24 inches below lowest adjacent finish pad grade. Continuous and isolated footings should have a minimum width of 12 inches and 24 inches, respectively. Retaining wall footings should be at least 24 inches wide.

BEARING CAPACITY: Spread footings supporting the proposed light exterior improvements may be designed for an allowable soil bearing pressure of 1,000 pounds per square foot (psf). This value may be increased by one-third for combinations of temporary loads such as those due to wind or seismic loads. Footings located within 10 feet from the face of slopes should be reviewed by this office.

FOOTING REINFORCING: Reinforcement requirements for foundations should be provided by a structural designer. However, based on the expected soil conditions, we recommend that the minimum reinforcing for continuous footings consist of at least 2 No. 5 bars positioned near the bottom of the footing and 2 No. 5 bars positioned near the top of the footing.

LATERAL LOAD RESISTANCE: Lateral loads against foundations may be resisted by friction between the bottom of the footing and the supporting soil, and by the passive pressure against the footing. The coefficient of friction between concrete and soil may be considered to be 0.25. The passive resistance may be considered to be equal to an equivalent fluid weight of 250 pounds per cubic foot. These values are based on the assumption that the footings are poured tight against undisturbed soil. If a combination of the passive pressure and friction is used, the friction value should be reduced by one-third.

FOUNDATION EXCAVATION OBSERVATION: All footing excavations should be observed by Christian Wheeler Engineering prior to placing of forms and reinforcing steel to determine whether the foundation recommendations presented herein are followed and that the foundation soils are as anticipated in the preparation of this report. All footing excavations should be excavated neat, level, and square. All loose or unsuitable material should be removed prior to the placement of concrete.

SETTLEMENT CHARACTERISTICS: The anticipated total and differential settlement is expected to be less than about 1 inch and 1 inch over 40 feet, respectively, provided the recommendations presented in this report are followed. It should be recognized that minor cracks normally occur in concrete slabs and foundations due to concrete shrinkage during curing or redistribution of stresses, therefore some cracks should be anticipated. Such cracks are not necessarily an indication of excessive vertical movements. However, it should be recognized that there is a higher degree of uncertainty in evaluating existing fills, and partially loading existing footings may result in increased differential settlements detrimental to the existing and proposed improvements. It is further our opinion that these conditions may result in cosmetic distress that may be easily repaired, and not result in significant structural distress to the structure.

EXPANSIVE CHARACTERISTICS: The prevailing foundation soils are assumed to have a low to expansive potential (EI between 21 and 90). The recommendations within this report reflect these conditions.

FOUNDATION PLAN REVIEW: The final foundation plan and accompanying details and notes should be submitted to this office for review. The intent of our review will be to verify that the plans used for construction reflect the minimum dimensioning and reinforcing criteria presented in this section and that no additional criteria are required due to changes in the foundation type or layout. It is not our intent to review structural plans, notes, details, or calculations to verify that the design engineer has correctly applied the geotechnical design values. It is the responsibility of the design engineer to properly design/specify the foundations and other structural elements based on the requirements of the structure and considering the information presented in this report.

SEISMIC DESIGN FACTORS

The seismic design factors applicable to the subject site are provided below. The seismic design factors were determined in accordance with the 2013 California Building Code. The site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters are presented in the following Table I.

TABLE I: SEISMIC DESIGN FACTORS

Site Coordinates: Latitude Longitude	32.822° -117.265°
Site Class	D
Site Coefficient F_a	1.0
Site Coefficient F_v	1.5
Spectral Response Acceleration at Short Periods S_s	1.302 g
Spectral Response Acceleration at 1 Second Period S_1	0.505 g
$S_{MS}=F_aS_s$	1.302 g
$S_{M1}=F_vS_1$	0.758 g
$S_{DS}=2/3*S_{MS}$	0.868 g
$S_{D1}=2/3*S_{M1}$	0.505 g

Probable ground shaking levels at the site could range from slight to moderate, depending on such factors as the magnitude of the seismic event and the distance to the epicenter. It is likely that the site will experience the effects of at least one moderate to large earthquake during the life of the proposed improvements.

ON-GRADE SLABS

GENERAL: It is recommended that the floor system of the proposed structure consist of a structural concrete slab or raised wood floors. Structural slab recommendations should be provided by the project structural engineer.

UNDER-SLAB VAPOR RETARDERS: Steps should be taken to minimize the transmission of moisture vapor from the subsoil through the interior slabs where it can potentially damage the interior floor coverings. Local industry standards typically include the placement of a vapor retarder, such as plastic, in a layer of coarse sand placed directly beneath the concrete slab. Two inches of sand are typically used above and below the plastic. The vapor retarder should be at least 15-mil Stegowrap® or similar material with sealed seams and should extend at least 12 inches down the sides of the interior and perimeter footings. The sand should have a sand equivalent of at least 30, and contain less than 10% passing the Number 100 sieve and less than 5% passing the Number 200 sieve. The membrane should be placed in accordance with the recommendation and consideration of ACI 302, "Guide for Concrete Floor and Slab Construction" and ASTM E1643, "Standards Practice for Installation of Water Vapor Retarder Used in Contact with Earth or Granular Fill Under Concrete Slabs." It is the flooring contractor's responsibility to place floor coverings in accordance with the flooring manufacturer specifications.

EXTERIOR CONCRETE FLATWORK: Exterior concrete slabs on grade should have a minimum thickness of 5 inches and be reinforced with at least No. 4 bars placed at 18 inches on center each way (ocew). Driveway slabs should have a minimum thickness of 5 inches and be reinforced with at least No. 4 bars placed at 12 inches ocew. Driveway slabs should be provided with a thickened edge a least 12 inches deep and 6 inches wide. All slabs should be provided with weakened plane joints in accordance with the American Concrete Institute (ACI) guidelines. Special attention should be paid to the method of concrete curing to reduce the potential for excessive shrinkage cracking. It should be recognized that minor cracks occur normally in concrete slabs due to shrinkage. Some shrinkage cracks should be expected and are not necessarily an indication of excessive movement or structural distress. However, it should be recognized that soils with medium (EI between 51 and 90) expansion potential may detrimentally affect light weight exterior improvements such as site walls, sidewalks, and driveways. Some distress to exterior improvements requiring future maintenance or even replacement should be anticipated due to expansive soils.

EARTH RETAINING WALLS

FOUNDATIONS: Foundations for any proposed retaining walls should be constructed in accordance with the foundation recommendations presented previously in this report.

PASSIVE PRESSURE: The passive pressure for the anticipated foundation soils may be considered to be 250 pounds per square foot per foot of depth. The upper foot of embedment should be neglected when calculating passive pressures, unless the foundation abuts a hard surface such as a concrete slab. The passive pressure may be increased by one-third for seismic loading. The coefficient of friction for concrete to soil may be assumed to be 0.25 for the resistance to lateral movement. When combining frictional and passive resistance, the friction should be reduced by one-third.

ACTIVE PRESSURE: The active soil pressure for the design of “unrestrained” and “restrained” earth retaining structures with level backfill may be assumed to be equivalent to the pressure of a fluid weighing 40 and 61 pounds per cubic foot, respectively. These pressures do not consider any other surcharge. If any are anticipated, this office should be contacted for the necessary increase in soil pressure. These values are based on a drained backfill condition.

Seismic lateral earth pressures may be assumed to equal an inverted triangle starting at the bottom of the wall with the maximum pressure equal to 11.5H pounds per square foot (where H = wall height in feet) occurring at the top of the wall.

WATERPROOFING AND WALL DRAINAGE SYSTEMS: The need for waterproofing should be evaluated by others. If required, the project architect should provide (or coordinate) waterproofing details for the retaining walls. The design values presented above are based on a drained backfill condition and do not consider hydrostatic pressures. The retaining wall designer should provide a detail for a wall drainage system. Typical retaining wall drain system details are presented as Plate No.

4 of this report for informational purposes. Additionally, outlets points for the retaining wall drain system should be coordinated with the project civil engineer.

BACKFILL: Retaining wall backfill soils should be compacted to at least 90 percent relative compaction. Expansive or clayey soils should not be used for backfill material. The wall should not be backfilled until the masonry has reached an adequate strength.

LIMITATIONS

REVIEW, OBSERVATION AND TESTING

The recommendations presented in this report are contingent upon our review of final plans and specifications. Such plans and specifications should be made available to the geotechnical engineer and engineering geologist so that they may review and verify their compliance with this report and with the California Building Code.

It is recommended that Christian Wheeler Engineering be retained to provide continuous soil engineering services during the earthwork operations. This is to verify compliance with the design concepts, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

UNIFORMITY OF CONDITIONS

The recommendations and opinions expressed in this report reflect our best estimate of the project requirements based on an evaluation of the subsurface soil conditions encountered at the subsurface exploration locations and on the assumption that the soil conditions do not deviate appreciably from those encountered. It should be recognized that the performance of the foundations and/or cut and fill slopes may be influenced by undisclosed or unforeseen variations in the soil conditions that may occur in the intermediate and unexplored areas. Any unusual conditions not covered in this report that may be encountered during site development should be brought to the attention of the geotechnical engineer so that he may make modifications if necessary.

CHANGE IN SCOPE

This office should be advised of any changes in the project scope or proposed site grading so that we may determine if the recommendations contained herein are appropriate. This should be verified in writing or modified by a written addendum.

TIME LIMITATIONS

The findings of this report are valid as of this date. Changes in the condition of a property can, however, occur with the passage of time, whether they be due to natural processes or the work of man on this or adjacent properties. In addition, changes in the Standards-of-Practice and/or Government Codes may occur. Due to such changes, the findings of this report may be invalidated wholly or in part by changes beyond our control. Therefore, this report should not be relied upon after a period of two years without a review by us verifying the suitability of the conclusions and recommendations.

PROFESSIONAL STANDARD

In the performance of our professional services, we comply with that level of care and skill ordinarily exercised by members of our profession currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the locations where our borings, surveys, and explorations are made, and that our data, interpretations, and recommendations be based solely on the information obtained by us. We will be responsible for those data, interpretations, and recommendations, but shall not be responsible for the interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty of any kind whatsoever, express or implied, is made or intended in connection with the work performed or to be performed by us, or by our proposal for consulting or other services, or by our furnishing of oral or written reports or findings.

CLIENT'S RESPONSIBILITY

It is the responsibility of the Clients, or his representatives, to ensure that the information and recommendations contained herein are brought to the attention of the structural engineer and architect for the project and incorporated into the project's plans and specifications. It is further their responsibility to take the necessary measures to insure that the contractor and his subcontractors carry out such recommendations during construction.

FIELD EXPLORATIONS

Four subsurface explorations were made on July 29, 2016 at the locations indicated on the Site Plan and Geotechnical Map included herewith as Plate No. 1. These explorations consisted of three borings drilled utilizing a portable drill rig and one hand dug test pit. The fieldwork was conducted under the observation and direction of our engineering geology personnel.

The explorations were carefully logged when made. The trench logs are presented on Appendix A. The soils are described in accordance with the Unified Soils Classification. In addition, a verbal textural description, the wet color, the apparent moisture, and the density or consistency is provided. The density of granular soils is given as very loose, loose, medium dense, dense or very dense. The consistency of silts or clays is given as either very soft, soft, medium stiff, stiff, very stiff, or hard.

Relatively undisturbed drive samples were collected using a modified California sampler. The sampler, with an external diameter of 3.0 inches, is lined with 1-inch long, thin, brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a 140-pound hammer falling 30 inches in general accordance with ASTM D 3550-84. The driving weight is permitted to fall freely. The number of blows per foot of driving, or as indicated, are presented on the boring logs as an index to the relative resistance of the sampled materials. The samples were removed from the sample barrel in the brass rings, and sealed. Relatively undisturbed chunk samples and bulk samples of the earth materials encountered were also collected. Samples were transported to our laboratory for testing.

LABORATORY TESTING

Laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. A brief description of the tests performed and the subsequent results are presented in Appendix B.

CWE LEGEND	
B-3	APPROXIMATE BORING LOCATION
P-1	APPROXIMATE TEST PIT LOCATION
Qaf / Ta	ARTIFICIAL FILL OVER ARDATH SHALE
F/C	APPROXIMATE FILL-CUT TRANSITION LINE

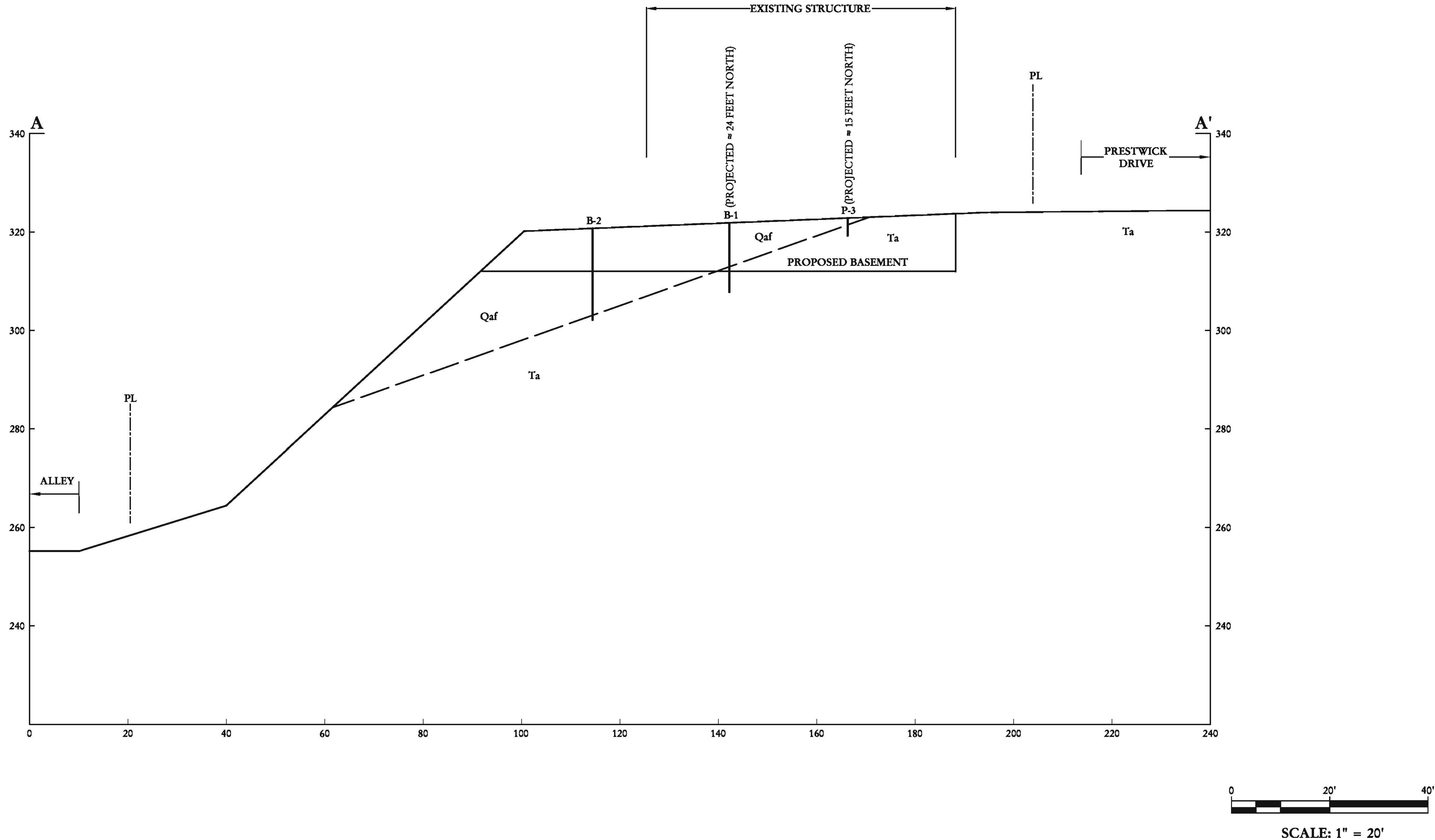
W+E

NOT TO SCALE

HERSHFIELD RESIDENCE
8230 PRESTWICK DRIVE
LA JOLLA, CALIFORNIA

DATE:	NOVEMBER 2016	JOB NO.:	2160443.02
BY:	SRD	PLATE NO.:	1

CHRISTIAN WHEELER
ENGINEERING

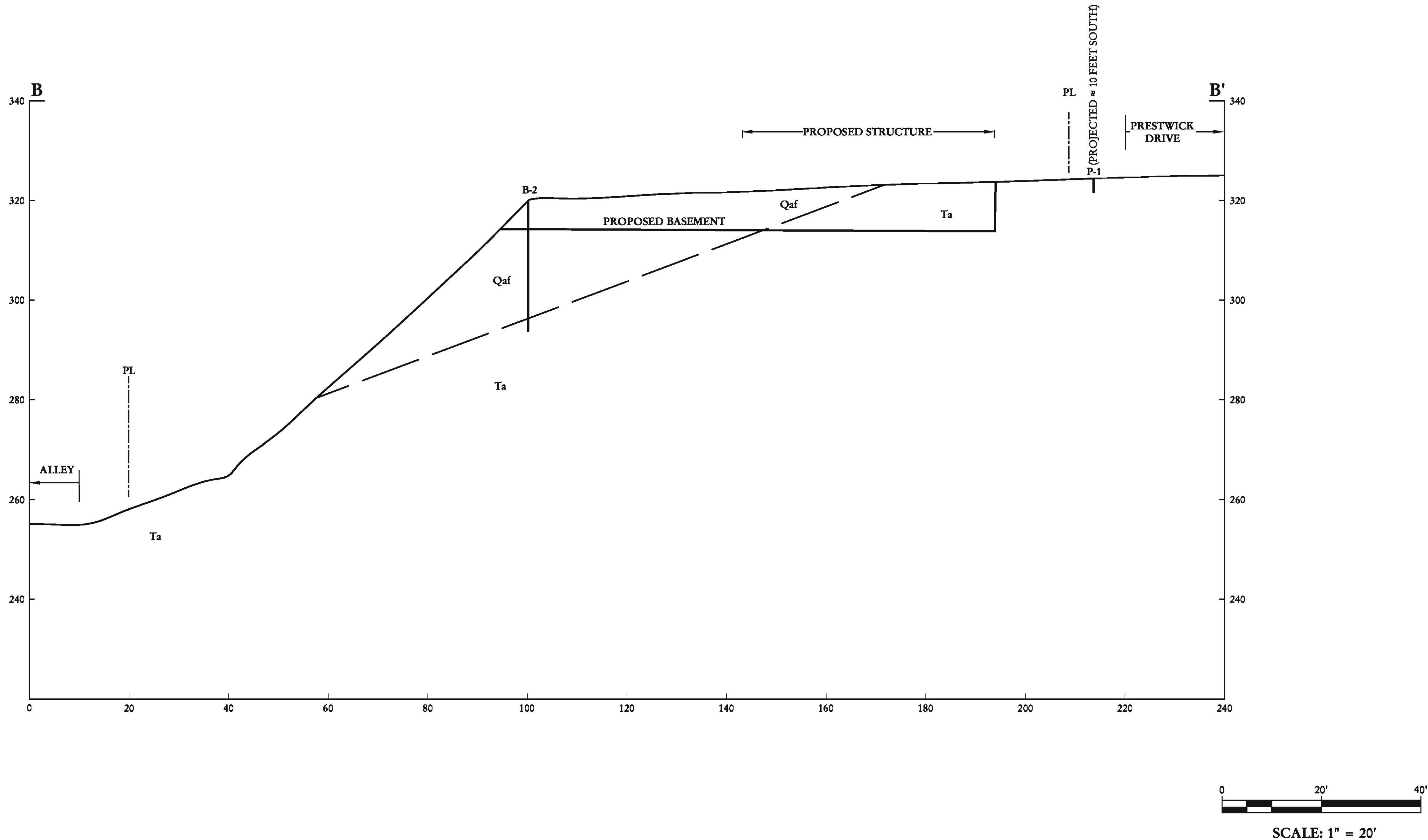


GEOLOGIC CROSS SECTION A-A'

HERSHFIELD RESIDENCE
8230 PRESTWICK DRIVE
SAN DIEGO, CALIFORNIA

DATE:	NOVEMBER 2016	JOB NO.:	2160443.02
BY:	SD	PLATE NO.:	2

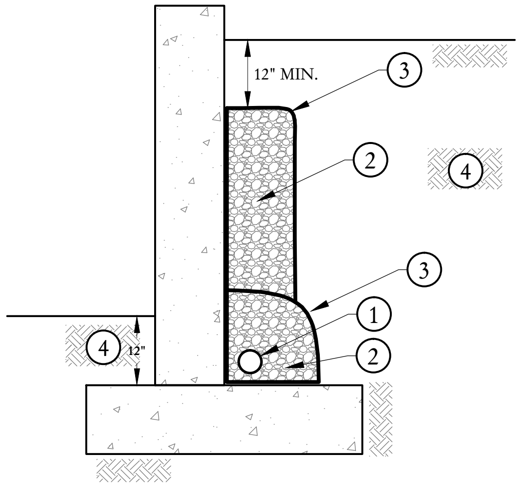
CHRISTIAN WHEELER ENGINEERING



GEOLOGIC CROSS SECTION B-B'

HERSHFIELD RESIDENCE 8230 PRESTWICK DRIVE SAN DIEGO, CALIFORNIA			
DATE:	NOVEMBER 2016	JOB NO.:	2160443.02
BY:	SD	PLATE NO.:	3

CHRISTIAN WHEELER
ENGINEERING



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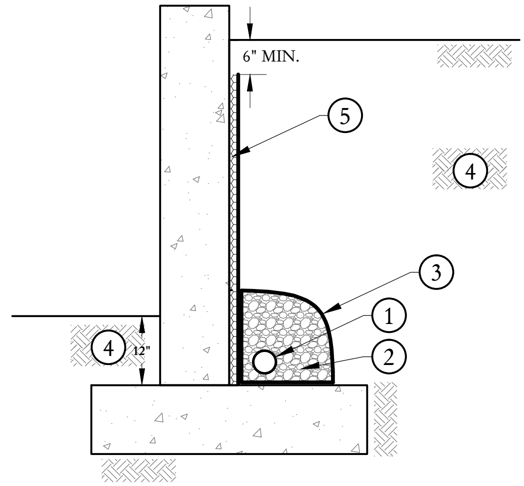

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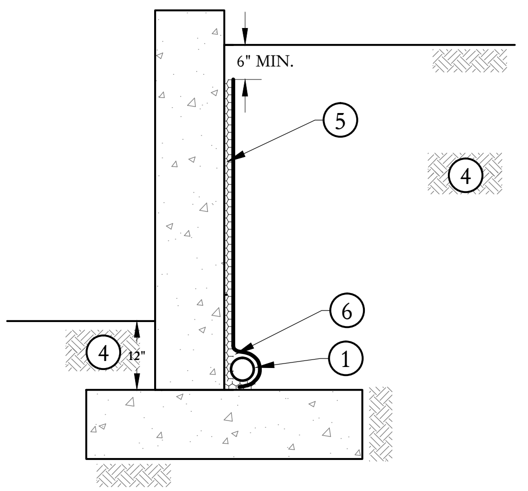

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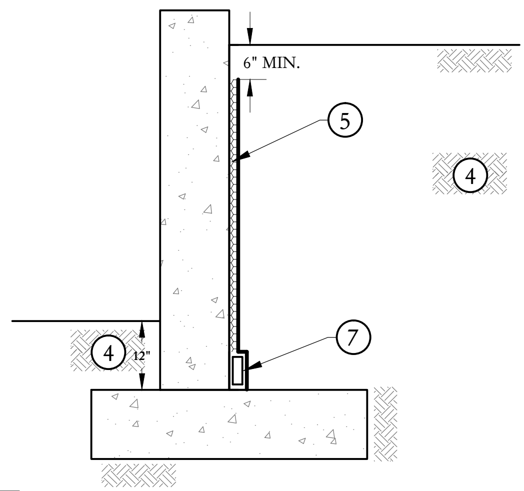

4 DETAIL

NOTES AND DETAILS

GENERAL NOTES:

1) THE NEED FOR WATERPROOFING SHOULD BE EVALUATED BY OTHERS.
2) WATERPROOFING TO BE DESIGNED BY OTHERS (CWE CAN PROVIDE A DESIGN IF REQUESTED).
3) EXTEND DRAIN TO SUITABLE DISCHARGE POINT PER CIVIL ENGINEER.
4) DO NOT CONNECT SURFACE DRAINS TO SUBDRAIN SYSTEM.

DETAILS:

1. 4-INCH PERFORATED PVC PIPE ON TOP OF FOOTING, HOLES POSITIONED DOWNWARD (SDR 35, SCHEDULE 40, OR EQUIVALENT).
2. ¾ INCH OPEN-GRADED CRUSHED AGGREGATE.
3. GEOFARBRIC WRAPPED COMPLETELY AROUND ROCK.
4. PROPERLY COMPACTED BACKFILL SOIL.
5. WALL DRAINAGE PANELS (MIRADRAIN OR EQUIVALENT) PLACED PER MANUFACTURER'S REC'S.
6. UNDERLAY SUBDRAIN WITH AND CUT FABRIC BACK FROM DRAINAGE PANELS AND WRAP FABRIC AROUND PIPE.
7. COLLECTION DRAIN (TOTAL DRAIN OR EQUIVALENT) LOCATED AT BASE OF WALL DRAINAGE PANEL PER MANUFACTURER'S RECOMMENDATIONS.

CANTILEVER RETAINING WALL DRAINAGE SYSTEMS	**HERSHFIELD RESIDENCE** 8230 PRESTWICK DRIVE SAN DIEGO, CALIFORNIA		CHRISTIAN WHEELER ENGINEERING
	DATE: NOVEMBER 2016	JOB NO.: 2160443.02	
	BY: SRD	PLATE NO.: 4	

Appendix A

Subsurface Explorations

LOG OF TEST PIT P-1

Sample Type and Laboratory Test Legend

Cal	Modified California Sampler	CK	Chunk
SPT	Standard Penetration Test	DR	Drive Ring
ST	Shelby Tube		
MD	Max Density	DS	Direct Shear
SO4	Soluble Sulfates	Con	Consolidation
SA	Sieve Analysis	EI	Expansion Index
HA	Hydrometer	R-Val	Resistance Value
SE	Sand Equivalent	Chl	Soluble Chlorides
PI	Plasticity Index	Res	pH & Resistivity
CP	Collapse Potential	SD	Sample Density

Date Logged:	7/29/16	Equipment:	Hand Tools
Logged By:	DJF	Auger Type:	N/A
Existing Elevation:	Unknown	Drive Type:	N/A
Finish Elevation:	Unknown	Depth to Water:	N/A

DEPTH (ft)	ELEVATION (ft)	GRAPHIC LOG	USCS SYMBOL	SUMMARY OF SUBSURFACE CONDITIONS (based on Unified Soil Classification System)	PENETRATION (blows per foot)	SAMPLE TYPE	BULK	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	RELATIVE COMPACTION (%)	LABORATORY TESTS
0				<u>Lawn and Associated Topsoil.</u>							
0.5			CL	<u>Artificial Fill (Qaf)</u>: Brown, very moist, soft, SILTY CLAY with SAND.							
1			CL	<u>Ardath Shale</u>: Light gray to yellowish-brown, moist, very stiff, SILTY CLAY, moderately fractured to 1.75'.							
1.5				Hard.		CK					
2											
2.5											
3				Test pit terminated at 2.5 feet. No groundwater or seepage encountered.							
3.5											
4											
4.5											
5											
5.5											
6											
6.5											
7											
7.5											

Notes:

Symbol Legend

- ∇ Groundwater Level During Drilling
- ▼ Groundwater Level After Drilling
- ❟❟ Apparent Seepage
- * No Sample Recovery
- ** Non-Representative Blow Count (rocks present)

HERSHFIELD RESIDENCE
8230 PRESTWICK DRIVE
LA JOLLA, CALIFORNIA

DATE:	DECEMBER 2016	JOB NO.:	2160443.02
BY:	SRD	APPENDIX A:	A-1

CHRISTIAN WHEELER ENGINEERING

LOG OF TEST BORING B-1

Sample Type and Laboratory Test Legend

Cal	Modified California Sampler	CK	Chunk
SPT	Standard Penetration Test	DR	Drive Ring
ST	Shelby Tube		
MD	Max Density	DS	Direct Shear
SO4	Soluble Sulfates	Con	Consolidation
SA	Sieve Analysis	EI	Expansion Index
HA	Hydrometer	R-Val	Resistance Value
SE	Sand Equivalent	Chl	Soluble Chlorides
PI	Plasticity Index	Res	pH & Resistivity
CP	Collapse Potential	SD	Sample Density

Date Logged:	7/29/16	Equipment:	Tripod
Logged By:	DJF	Auger Type:	6 inch Solid Flight
Existing Elevation:	Unknown	Drive Type:	140lbs/30 inches
Proposed Elevation:	Unknown	Depth to Water:	N/A

DEPTH (ft)	ELEVATION (ft)	GRAPHIC LOG	USCS SYMBOL	SUMMARY OF SUBSURFACE CONDITIONS (based on Unified Soil Classification System)	PENETRATION (blows per foot)	SAMPLE TYPE	BULK	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	RELATIVE COMPACTION (%)	LABORATORY TESTS
0			SM	**Artificial Fill (Qaf):** Brown to grayish-brown, very moist, loose, fine- to coarse-grained, SILTY SAND with gravels.							
				Brown, moist, medium dense.	20	Cal					
			CL	Yellowish-brown to olive brown, moist to very moist, medium stiff, SILTY CLAY.							
5					10	Cal					
10					8	Cal					
				Stiff.							
15					25	Cal					
			ML-CL	**Ardath Shale (Ta):** Light gray to yellowish-brown, moist, hard, CLAYEY SILT-SILTY CLAY.	50/4"	Cal					
				Boring terminated at 18.5 feet. No groundwater or seepage encountered.							
20											
25											
30											

Notes:

Symbol Legend

- Groundwater Level During Drilling
- Groundwater Level After Drilling
- Apparent Seepage
- * No Sample Recovery
- ** Non-Representative Blow Count (rocks present)

HERSHFIELD RESIDENCE
8230 PRESTWICK DRIVE
LA JOLLA, CALIFORNIA

DATE:	DECEMBER 2016	JOB NO.:	2160443.02
BY:	SRD	APPENDIX A:	A-2

CHRISTIAN WHEELER
ENGINEERING

LOG OF TEST BORING B-2

Date Logged:	7/29/16	Equipment:	Tripod
Logged By:	DJF	Auger Type:	6 inch Solid Flight
Existing Elevation:	Unknown	Drive Type:	140lbs/30 inches
Proposed Elevation:	Unknown	Depth to Water:	N/A

Sample Type and Laboratory Test Legend

Cal	Modified California Sampler	CK	Chunk
SPT	Standard Penetration Test	DR	Drive Ring
ST	Shelby Tube		
MD	Max Density	DS	Direct Shear
SO4	Soluble Sulfates	Con	Consolidation
SA	Sieve Analysis	EI	Expansion Index
HA	Hydrometer	R-Val	Resistance Value
SE	Sand Equivalent	Chl	Soluble Chlorides
PI	Plasticity Index	Res	pH & Resistivity
CP	Collapse Potential	SD	Sample Density

DEPTH (ft)	ELEVATION (ft)	GRAPHIC LOG	USCS SYMBOL	SUMMARY OF SUBSURFACE CONDITIONS (based on Unified Soil Classification System)	PENETRATION (blows per foot)	SAMPLE TYPE	BULK	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	RELATIVE COMPACTION (%)	LABORATORY TESTS
0			CL	**Artificial Fill (Qaf):** Yellowish-brown to olive brown, very moist, soft, SILTY CLAY.							
				Medium stiff.							
5				Crushed rock in sample associated with adjacent retaining wall.							
					10	Cal*					
10				Stiff.							
					24	Cal					
15											
					29	Cal					
20				Olive brown to yellowish-brown, moist.							
25			ML-CL	**Ardath Shale (Ta):** Light gray to yellowish-brown, moist, hard, CLAYEY SILT-SILTY CLAY.	50/6"	Cal					
				Boring terminated at 25.5 feet. No groundwater or seepage encountered.							
30											

Notes:

Symbol Legend

- Groundwater Level During Drilling
- Groundwater Level After Drilling
- Apparent Seepage
- * No Sample Recovery
- ** Non-Representative Blow Count (rocks present)

HERSHFIELD RESIDENCE
8230 PRESTWICK DRIVE
LA JOLLA, CALIFORNIA

DATE:	DECEMBER 2016	JOB NO.:	2160443.02
BY:	SRD	APPENDIX A:	A-3

CHRISTIAN WHEELER ENGINEERING

LOG OF TEST BORING B-3

Sample Type and Laboratory Test Legend

Cal	Modified California Sampler	CK	Chunk
SPT	Standard Penetration Test	DR	Drive Ring
ST	Shelby Tube		
MD	Max Density	DS	Direct Shear
SO4	Soluble Sulfates	Con	Consolidation
SA	Sieve Analysis	EI	Expansion Index
HA	Hydrometer	R-Val	Resistance Value
SE	Sand Equivalent	Chl	Soluble Chlorides
PI	Plasticity Index	Res	pH & Resistivity
CP	Collapse Potential	SD	Sample Density

Date Logged:	7/29/16	Equipment:	Tripod
Logged By:	DJF	Auger Type:	6 inch Solid Flight
Existing Elevation:	Unknown	Drive Type:	140lbs/30 inches
Proposed Elevation:	Unknown	Depth to Water:	N/A

DEPTH (ft)	ELEVATION (ft)	GRAPHIC LOG	USCS SYMBOL	SUMMARY OF SUBSURFACE CONDITIONS (based on Unified Soil Classification System)	PENETRATION (blows per foot)	SAMPLE TYPE	BULK	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	RELATIVE COMPACTION (%)	LABORATORY TESTS
0			CL	**Artificial Fill (Qaf):** Olive brown, very moist, soft, SILTY CLAY.							
				Medium stiff.							
5											
					10	Cal					
10					12	Cal					
				Seepage at 12 to 13 feet above contact with Ardath Shale.							
			CL	**Ardath Shale (Ta):** Light gray to yellowish-brown, moist, hard, SILTY CLAY, fractured/moderately weathered to 13.5 feet.	50/6"	Cal					
15				Boring terminated at 14.5 feet. Seepage encountered at 12 feet.							
20											
25											
30											

Notes:

Symbol Legend

- Groundwater Level During Drilling
- Groundwater Level After Drilling
- Apparent Seepage
- * No Sample Recovery
- ** Non-Representative Blow Count (rocks present)

HERSHFIELD RESIDENCE
8230 PRESTWICK DRIVE
LA JOLLA, CALIFORNIA

DATE:	DECEMBER 2016	JOB NO.:	2160443.02
BY:	SRD	APPENDIX A:	A-4

CHRISTIAN WHEELER
ENGINEERING

Appendix B

Laboratory Test Results

Laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. Brief descriptions of the tests performed are presented below:

a) **CLASSIFICATION:** Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soil Classification System and are presented on the exploration logs in Appendix A.

b) **MOISTURE-DENSITY: MOISTURE-DENSITY:** In-place moisture contents and dry densities were determined for selected soil samples in accordance with ATM D 1188 and D 2937. The results are summarized in the test pit and boring logs presented in Appendix A.

c) **DIRECT SHEAR:** Direct shear tests were performed on selected samples of the on-site soils in accordance with ASTM D 3080.

d) **EXPANSION INDEX TEST:** Expansion index tests were performed on a selected remolded soil sample in accordance with ASTM D 4829.

e) **GRAIN SIZE DISTRIBUTION:** The grain size distribution of selected samples was determined in accordance with ASTM C136 and/or ASTM D 422.

f) **SOLUBLE SULFATES:** The soluble sulfate content of selected soil samples was determined in accordance with California Test Method 417.

LABORATORY TEST RESULTS

PROPOSED HERSHFIELD RESIDENCE

8230 PRESTWICK DRIVE

LA JOLLA, CALIFORNIA

MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT (ASTM D1557)

Sample Location	**Boring B-2 @ 0-5'**
Sample Description	Yellow Silty Sand (SM)
Maximum Density	121.8 pcf
Optimum Moisture	11.7 %

DIRECT SHEAR (ASTM D3080)

Sample Location	**Test Pit P-1 @ 1'-2½'**	**Boring B-2 @ 0-5'**	**Boring B-2 @ 16½'**
Sample Type	Remolded -In-Situ Density & Moisture Content	Remolded to 90%	Undisturbed
Friction Angle	31°	26°	30°
Cohesion	500 psf	400 psf	750 psf

EXPANSION INDEX TESTS (ASTM D4829)

Sample Location	**Boring B-2 @ 0-5'**
Initial Moisture:	10.6 %
Initial Dry Density	108.3 pcf
Final Moisture:	22.3 %
Expansion Index:	46 (Low)

GRAIN SIZE DISTRIBUTION (ASTM D422)

Sample Location	**Test Pit P-1 @ 1'-2½'**	**Boring B-2 @ 0-5'**	**Boring B-2 @ 25½'**
Sieve Size	*Percent Passing*	*Percent Passing*	
1"		100	
¾"		99	
½"		96	
⅜		93	
#4	100	86	100
#8	97	84	99
#16	95	83	98
#30	93	81	97
#50	92	74	97
#100	90	69	96
#200	85	64	92
0.05 mm	77	60	88
0.005 mm	25	37	31
0.001 mm	13	23	6

SOLUBLE SULFATES (CALIFORNIA TEST 417)

Sample Location	**Boring B-2 @ 0-5'**	**Test Pit P-1 @ 1'-2½'**
Soluble Sulfate	0.144 % (SO_4)	0.017 % (SO_4)

Appendix C

References

REFERENCES

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Appendix D

Recommended Grading Specifications – General Provisions

RECOMMENDED GRADING SPECIFICATIONS - GENERAL PROVISIONS

PROPOSED HERSHFIELD REISDENCE

8230 PRESTWICK DRIVE

LA JOLLA, CALIFORNIA

GENERAL INTENT

The intent of these specifications is to establish procedures for clearing, compacting natural ground, preparing areas to be filled, and placing and compacting fill soils to the lines and grades shown on the accepted plans. The recommendations contained in the preliminary geotechnical investigation report and/or the attached Special Provisions are a part of the Recommended Grading Specifications and shall supersede the provisions contained hereinafter in the case of conflict. These specifications shall only be used in conjunction with the geotechnical report for which they are a part. No deviation from these specifications will be allowed, except where specified in the geotechnical report or in other written communication signed by the Geotechnical Engineer.

OBSERVATION AND TESTING

Christian Wheeler Engineering shall be retained as the Geotechnical Engineer to observe and test the earthwork in accordance with these specifications. It will be necessary that the Geotechnical Engineer or his representative provide adequate observation so that he may provide his opinion as to whether or not the work was accomplished as specified. It shall be the responsibility of the contractor to assist the Geotechnical Engineer and to keep him appraised of work schedules, changes and new information and data so that he may provide these opinions. In the event that any unusual conditions not covered by the special provisions or preliminary geotechnical report are encountered during the grading operations, the Geotechnical Engineer shall be contacted for further recommendations.

If, in the opinion of the Geotechnical Engineer, substandard conditions are encountered, such as questionable or unsuitable soil, unacceptable moisture content, inadequate compaction, adverse weather, etc., construction should be stopped until the conditions are remedied or corrected or he shall recommend rejection of this work.

Tests used to determine the degree of compaction should be performed in accordance with the following American Society for Testing and Materials test methods:

Maximum Density & Optimum Moisture Content - ASTM D1557
Density of Soil In-Place - ASTM D1556 or ASTM D6938

All densities shall be expressed in terms of Relative Compaction as determined by the foregoing ASTM testing procedures.

PREPARATION OF AREAS TO RECEIVE FILL

All vegetation, brush and debris derived from clearing operations shall be removed, and legally disposed of. All areas disturbed by site grading should be left in a neat and finished appearance, free from unsightly debris.

After clearing or benching the natural ground, the areas to be filled shall be scarified to a depth of 6 inches, brought to the proper moisture content, compacted and tested for the specified minimum degree of compaction. All loose soils in excess of 6 inches thick should be removed to firm natural ground which is defined as natural soil which possesses an in-situ density of at least 90 percent of its maximum dry density.

When the slope of the natural ground receiving fill exceeds 20 percent (5 horizontal units to 1 vertical unit), the original ground shall be stepped or benched. Benches shall be cut to a firm competent formational soil. The lower bench shall be at least 10 feet wide or 1-1/2 times the equipment width, whichever is greater, and shall be sloped back into the hillside at a gradient of not less than two (2) percent. All other benches should be at least 6 feet wide. The horizontal portion of each bench shall be compacted prior to receiving fill as specified herein for compacted natural ground. Ground slopes flatter than 20 percent shall be benched when considered necessary by the Geotechnical Engineer.

Any abandoned buried structures encountered during grading operations must be totally removed. All underground utilities to be abandoned beneath any proposed structure should be removed from within 10 feet of the structure and properly capped off. The resulting depressions from the above described procedure should be backfilled with acceptable soil that is compacted to the requirements of the Geotechnical Engineer. This includes, but is not limited to, septic tanks, fuel tanks, sewer lines or leach lines, storm drains and water lines. Any buried structures or utilities not to be abandoned should be brought to the attention of the Geotechnical Engineer so that he may determine if any special recommendation will be necessary.

All water wells which will be abandoned should be backfilled and capped in accordance to the requirements set forth by the Geotechnical Engineer. The top of the cap should be at least 4 feet below finish grade or 3 feet below the bottom of footing whichever is greater. The type of cap will depend on the diameter of the well and should be determined by the Geotechnical Engineer and/or a qualified Structural Engineer.

FILL MATERIAL

Materials to be placed in the fill shall be approved by the Geotechnical Engineer and shall be free of vegetable matter and other deleterious substances. Granular soil shall contain sufficient fine material to fill the voids. The definition and disposition of oversized rocks and expansive or detrimental soils are covered in the geotechnical report or Special Provisions. Expansive soils, soils of poor gradation, or soils with low strength characteristics may be thoroughly mixed with other soils to provide satisfactory fill material, but only with the explicit consent of the Geotechnical Engineer. Any import material shall be approved by the Geotechnical Engineer before being brought to the site.

PLACING AND COMPACTION OF FILL

Approved fill material shall be placed in areas prepared to receive fill in layers not to exceed 6 inches in compacted thickness. Each layer shall have a uniform moisture content in the range that will allow the compaction effort to be efficiently applied to achieve the specified degree of compaction. Each layer shall be uniformly compacted to the specified minimum degree of compaction with equipment of adequate size to economically compact the layer. Compaction equipment should either be specifically designed for soil compaction or of proven reliability. The minimum degree of compaction to be achieved is specified in either the Special Provisions or the recommendations contained in the preliminary geotechnical investigation report.

When the structural fill material includes rocks, no rocks will be allowed to nest and all voids must be carefully filled with soil such that the minimum degree of compaction recommended in the Special Provisions is achieved. The maximum size and spacing of rock permitted in structural fills and in non-structural fills is discussed in the geotechnical report, when applicable.

Field observation and compaction tests to estimate the degree of compaction of the fill will be taken by the Geotechnical Engineer or his representative. The location and frequency of the tests shall be at the Geotechnical Engineer's discretion. When the compaction test indicates that a particular layer is at less than the required degree of compaction, the layer shall be reworked to the satisfaction of the Geotechnical Engineer and until the desired relative compaction has been obtained.

Fill slopes shall be compacted by means of sheepsfoot rollers or other suitable equipment. Compaction by sheepsfoot roller shall be at vertical intervals of not greater than four feet. In addition, fill slopes at a ratio of two horizontal to one vertical or flatter, should be trackrolled. Steeper fill slopes shall be over-built and cut-back to finish contours after the slope has been constructed. Slope compaction operations shall result in all fill material six or more inches inward from the finished face of the slope having a relative compaction of at least 90 percent of maximum dry density or the degree of compaction specified in the Special Provisions section of this specification. The compaction operation on the slopes shall be continued until the Geotechnical Engineer is of the opinion that the slopes will be surficially stable.

Density tests in the slopes will be made by the Geotechnical Engineer during construction of the slopes to determine if the required compaction is being achieved. Where failing tests occur or other field problems arise, the Contractor will be notified that day of such conditions by written communication from the Geotechnical Engineer or his representative in the form of a daily field report.

If the method of achieving the required slope compaction selected by the Contractor fails to produce the necessary results, the Contractor shall rework or rebuild such slopes until the required degree of compaction is obtained, at no cost to the Owner or Geotechnical Engineer.

CUT SLOPES

The Engineering Geologist shall inspect cut slopes excavated in rock or lithified formational material during the grading operations at intervals determined at his discretion. If any conditions not anticipated in the preliminary report such as perched water, seepage, lenticular or confined strata of a potentially adverse nature, unfavorably inclined bedding, joints or fault planes are encountered during grading, these conditions shall be analyzed by the Engineering Geologist and Geotechnical Engineer to determine if mitigating measures are necessary.

Unless otherwise specified in the geotechnical report, no cut slopes shall be excavated higher or steeper than that allowed by the ordinances of the controlling governmental agency.

ENGINEERING OBSERVATION

Field observation by the Geotechnical Engineer or his representative shall be made during the filling and compaction operations so that he can express his opinion regarding the conformance of the grading with acceptable standards of practice. Neither the presence of the Geotechnical Engineer or his representative or the observation and testing shall release the Grading Contractor from his duty to compact all fill material to the specified degree of compaction.

SEASON LIMITS

Fill shall not be placed during unfavorable weather conditions. When work is interrupted by heavy rain, filling operations shall not be resumed until the proper moisture content and density of the fill materials can be achieved. Damaged site conditions resulting from weather or acts of God shall be repaired before acceptance of work.

RECOMMENDED GRADING SPECIFICATIONS - SPECIAL PROVISIONS

RELATIVE COMPACTION: The minimum degree of compaction to be obtained in compacted natural ground, compacted fill, and compacted backfill shall be at least 90 percent. For street and parking lot subgrade, the upper six inches should be compacted to at least 95 percent relative compaction.

EXPANSIVE SOILS: Detrimentally expansive soil is defined as clayey soil which has an expansion index of 50 or greater when tested in accordance with the Uniform Building Code Standard 29-2.

OVERSIZED MATERIAL: Oversized fill material is generally defined herein as rocks or lumps of soil over 6 inches in diameter. Oversized materials should not be placed in fill unless recommendations of placement of such material are provided by the Geotechnical Engineer. At least 40 percent of the fill soils shall pass through a No. 4 U.S. Standard Sieve.

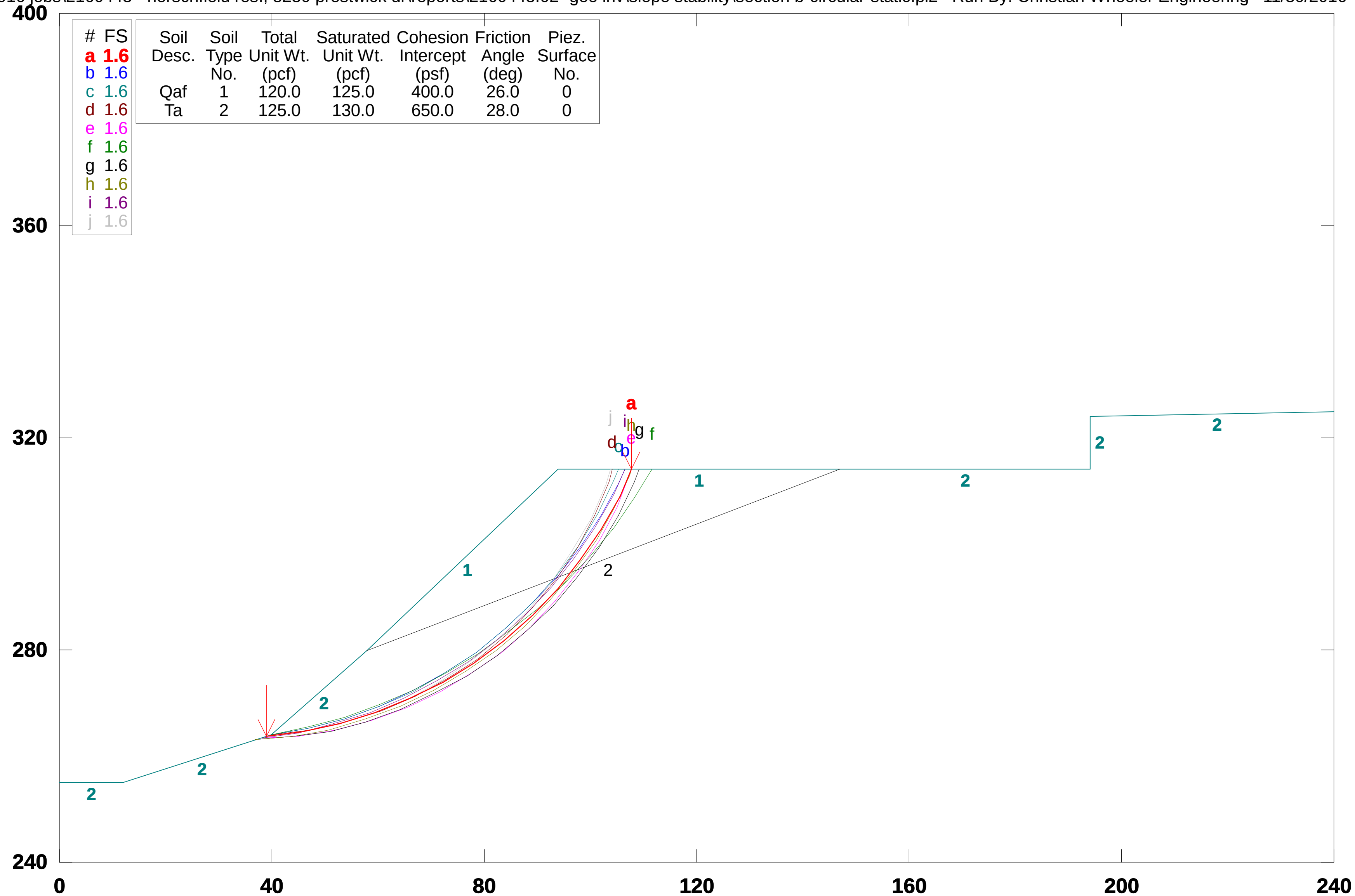
TRANSITION LOTS: Where transitions between cut and fill occur within the proposed building pad, the cut portion should be undercut a minimum of one foot below the base of the proposed footings and recompacted as structural backfill. In certain cases that would be addressed in the geotechnical report, special footing reinforcement or a combination of special footing reinforcement and undercutting may be required.

Appendix E

Global and Surficial Stability Analyses

Hershfield Res Section B

w:\2016 jobs\2160443 - herschfield res., 8230 prestwick dr\reports\2160443.02- geo inv\slope stability\section b-circular-static.pl2 Run By: Christian Wheeler Engineering 11/30/2016 02:11PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Qaf	1	120.0	125.0	400.0	26.0	0
Ta	2	125.0	130.0	650.0	28.0	0

GSTABL7 v.2 FSmin=1.6

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

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                              ***  GSTABL7  ***
                   ** GSTABL7 by Garry H. Gregory, P.E. **
       ** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **
                 (All Rights Reserved-Unauthorized Use Prohibited)
  *********************************************************************************
                      SLOPE STABILITY ANALYSIS SYSTEM
         Modified Bishop, Simplified Janbu, or GLE Method of Slices.
         (Includes Spencer & Morgenstern-Price Type Analysis)
         Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
         Nonlinear Undrained Shear Strength, Curved Phi Envelope,
         Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
         Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
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ts\2160443.02- Geo Inv\Slope Stability\Section B-Circular-Static.PLT
    PROBLEM DESCRIPTION:  Hershfield Res
                          Section B
    BOUNDARY COORDINATES
        8 Top   Boundaries
        9 Total Boundaries
    Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
       No.        (ft)       (ft)       (ft)       (ft)     Below Bnd
        1          0.00     255.00      12.00     255.00        2
        2         12.00     255.00      40.00     264.00        2
        3         40.00     264.00      58.00     280.00        2
        4         58.00     280.00      94.00     314.00        1
        5         94.00     314.00     147.00     314.00        1
        6        147.00     314.00     194.00     314.00        2
        7        194.00     314.00     194.10     324.00        2
        8        194.10     324.00     240.00     325.00        2
        9         58.00     280.00     147.00     314.00        2
    User Specified Y-Origin =      240.00(ft)
    Default X-Plus Value = 0.00(ft)
    Default Y-Plus Value = 0.00(ft)
   ISOTROPIC SOIL PARAMETERS
     2 Type(s) of Soil
    Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
    Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
     No.  (pcf)    (pcf)     (psf)     (deg)    Param.   (psf)     No.
      1   120.0    125.0     400.0     26.0    0.00       0.0      0
      2   125.0    130.0     650.0     28.0    0.00       0.0      0
    A Critical Failure Surface Searching Method, Using A Random
    Technique For Generating Circular Surfaces, Has Been Specified.
    3000 Trial Surfaces Have Been Generated.
     500 Surface(s) Initiate(s) From Each Of     6 Points Equally Spaced
    Along The Ground Surface Between  X =  35.00(ft)
                                and  X =  40.00(ft)
    Each Surface Terminates Between   X =  94.00(ft)
                               and   X = 194.00(ft)
    Unless Further Limitations Were Imposed, The Minimum Elevation
    At Which A Surface Extends Is  Y =      0.00(ft)
     7.00(ft) Line Segments Define Each Trial Failure Surface.
    Following Are Displayed The Ten Most Critical Of The Trial
          Failure Surfaces Evaluated. They Are
          Ordered - Most Critical First.
          * * Safety Factors Are Calculated By The Modified Bishop Method * *
          Total Number of Trial Surfaces Evaluated =  3000
          Statistical Data On All Valid FS Values:
             FS Max =   4.501   FS Min =   1.589   FS Ave =   2.984
             Standard Deviation =    0.826   Coefficient of Variation =   27.69 %
          Failure Surface Specified By 14 Coordinate Points
            Point      X-Surf      Y-Surf
```

No.	(ft)	(ft)
1	39.00	263.68
2	45.94	264.58
3	52.78	266.07
4	59.47	268.14
5	65.96	270.77
6	72.20	273.94
7	78.14	277.63
8	83.76	281.82
9	88.99	286.46
10	93.81	291.54
11	98.18	297.01
12	102.07	302.83
13	105.45	308.96
14	107.70	314.00

Circle Center At X = 31.94 ; Y = 345.37 ; and Radius = 82.00

Factor of Safety

*** 1.589 ***

Individual data on the 17 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	1.0	12.0	0.0	0.0	0.	0.	0.0	0.0	0.0
2	5.9	1816.3	0.0	0.0	0.	0.	0.0	0.0	0.0
3	6.8	5979.7	0.0	0.0	0.	0.	0.0	0.0	0.0
4	5.2	7046.8	0.0	0.0	0.	0.	0.0	0.0	0.0
5	1.5	2343.5	0.0	0.0	0.	0.	0.0	0.0	0.0
6	6.5	12075.6	0.0	0.0	0.	0.	0.0	0.0	0.0
7	6.2	13929.2	0.0	0.0	0.	0.	0.0	0.0	0.0
8	5.9	14901.6	0.0	0.0	0.	0.	0.0	0.0	0.0
9	5.6	15035.7	0.0	0.0	0.	0.	0.0	0.0	0.0
10	5.2	14409.3	0.0	0.0	0.	0.	0.0	0.0	0.0
11	4.8	13132.3	0.0	0.0	0.	0.	0.0	0.0	0.0
12	0.2	505.4	0.0	0.0	0.	0.	0.0	0.0	0.0
13	2.3	5695.9	0.0	0.0	0.	0.	0.0	0.0	0.0
14	1.9	4157.7	0.0	0.0	0.	0.	0.0	0.0	0.0
15	3.9	6574.1	0.0	0.0	0.	0.	0.0	0.0	0.0
16	3.4	3289.4	0.0	0.0	0.	0.	0.0	0.0	0.0
17	2.2	679.5	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	40.00	264.00
2	46.88	265.30
3	53.63	267.14
4	60.21	269.52
5	66.59	272.42
6	72.70	275.82
7	78.53	279.70
8	84.03	284.04
9	89.16	288.80
10	93.89	293.96
11	98.19	299.48
12	102.04	305.32
13	105.41	311.46
14	106.55	314.00

Circle Center At X = 27.39 ; Y = 350.07 ; and Radius = 86.99

Factor of Safety

*** 1.590 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	40.00	264.00
2	46.90	265.19
3	53.67	266.96
4	60.27	269.31
5	66.64	272.20
6	72.74	275.63
7	78.53	279.56

```
     8         83.97      283.97
     9         89.01      288.83
    10         93.62      294.10
    11         97.76      299.74
    12        101.41      305.71
    13        104.54      311.97
    14        105.35      314.00
Circle Center At X =    29.56 ; Y =   345.35 ; and Radius =    82.01
       Factor of Safety
      ***    1.591   ***
Failure Surface Specified By 14 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
     1         40.00      264.00
     2         46.95      264.82
     3         53.79      266.31
     4         60.46      268.44
     5         66.89      271.20
     6         73.03      274.57
     7         78.81      278.51
     8         84.19      282.99
     9         89.12      287.96
    10         93.55      293.38
    11         97.43      299.21
    12        100.74      305.38
    13        103.44      311.83
    14        104.11      314.00
Circle Center At X =    35.02 ; Y =   336.49 ; and Radius =    72.67
       Factor of Safety
      ***    1.593   ***
Failure Surface Specified By 15 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
     1         38.00      263.36
     2         44.98      263.82
     3         51.90      264.93
     4         58.67      266.68
     5         65.26      269.05
     6         71.60      272.02
     7         77.64      275.56
     8         83.32      279.64
     9         88.60      284.24
    10         93.44      289.30
    11         97.78      294.79
    12        101.60      300.66
    13        104.85      306.86
    14        107.52      313.33
    15        107.72      314.00
Circle Center At X =    36.47 ; Y =   338.83 ; and Radius =    75.48
       Factor of Safety
      ***    1.596   ***
Failure Surface Specified By 14 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
     1         40.00      264.00
     2         46.85      265.46
     3         53.58      267.37
     4         60.17      269.73
     5         66.58      272.54
     6         72.80      275.76
     7         78.77      279.40
     8         84.50      283.44
     9         89.93      287.85
    10         95.06      292.61
    11         99.85      297.72
    12        104.29      303.13
    13        108.35      308.83
    14        111.54      314.00
Circle Center At X =    21.90 ; Y =   366.07 ; and Radius =   103.66
       Factor of Safety
```

*** 1.597 ***

Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	37.00	263.04
2	43.98	263.59
3	50.88	264.74
4	57.66	266.49
5	64.26	268.82
6	70.64	271.71
7	76.73	275.15
8	82.51	279.10
9	87.93	283.54
10	92.94	288.43
11	97.50	293.73
12	101.59	299.42
13	105.17	305.43
14	108.22	311.73
15	109.08	314.00

Circle Center At X = 34.13 ; Y = 343.65 ; and Radius = 80.67

Factor of Safety

*** 1.597 ***

Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	37.00	263.04
2	43.97	263.72
3	50.85	265.00
4	57.59	266.87
5	64.15	269.32
6	70.47	272.32
7	76.51	275.86
8	82.23	279.91
9	87.57	284.43
10	92.50	289.40
11	96.98	294.78
12	100.98	300.52
13	104.47	306.59
14	107.43	312.94
15	107.81	314.00

Circle Center At X = 32.59 ; Y = 343.91 ; and Radius = 80.99

Factor of Safety

*** 1.598 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	38.00	263.36
2	44.92	264.39
3	51.74	266.00
4	58.39	268.17
5	64.85	270.88
6	71.05	274.11
7	76.97	277.85
8	82.56	282.06
9	87.78	286.72
10	92.60	291.80
11	96.98	297.27
12	100.89	303.07
13	104.31	309.18
14	106.50	314.00

Circle Center At X = 29.03 ; Y = 347.15 ; and Radius = 84.27

Factor of Safety

*** 1.599 ***

Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	39.00	263.68
2	45.95	264.54
3	52.78	266.05
4	59.45	268.19

5	65.88	270.94
6	72.03	274.28
7	77.84	278.19
8	83.26	282.62
9	88.24	287.54
10	92.74	292.90
11	96.72	298.66
12	100.14	304.77
13	102.97	311.17
14	103.92	314.00

Circle Center At X =   33.38 ; Y =   337.96 ; and Radius =   74.49

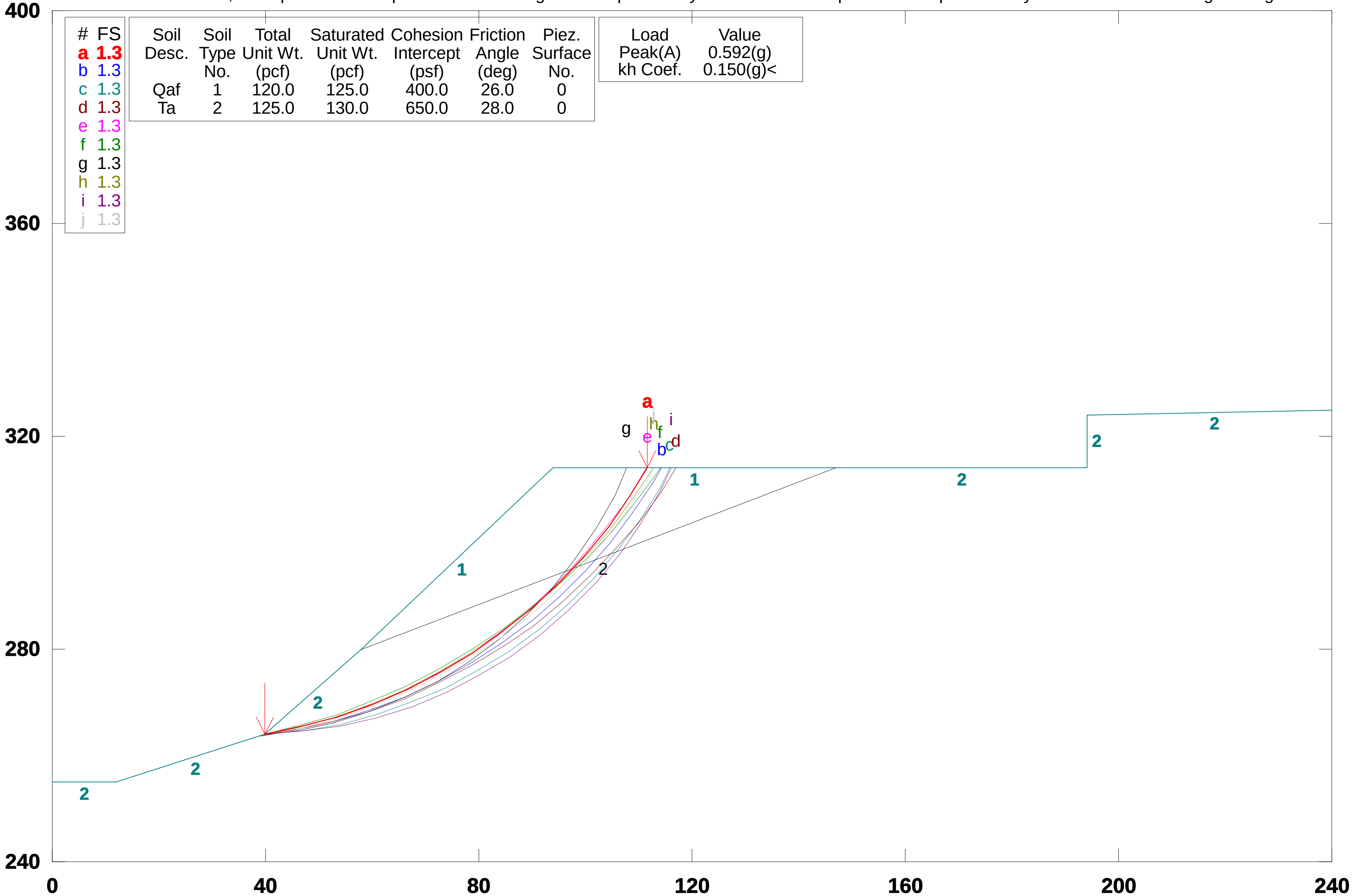
Factor of Safety

*** 1.600 ***

**** END OF GSTABL7 OUTPUT ****

Hershfield Res Section B

w:\2016 jobs\2160443 - herschfield res., 8230 prestwick dr\reports\2160443.02- geo inv\slope stability\section b-circular-pseudostatic.pl2 Run By: Christian Wheeler Engineering 11/30/2016 02:17PM



GSTABL7 v.2 FSmin=1.3

Safety Factors Are Calculated By The Modified Bishop Method

***** GSTABL7 *****

** GSTABL7 by Garry H. Gregory, P.E. **
** Original Version 1.0, January 1996; Current Version 2.003, June 2002 **

****
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
****

Analysis Run Date: 11/30/2016
Time of Run: 02:17PM
Run By: Christian Wheeler Engineering
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Output Filename: W:\2016 Jobs\2160443 - Herschfield Res., 8230 Prestwick Dr\Reports\2160443.02- Geo Inv\Slope Stability\Section B-Circular-PseudoStatic.OUT
Unit System: English
Plotted Output Filename: W:\2016 Jobs\2160443 - Herschfiees., 8230 Prestwick Dr\Reports\2160443.02- Geo Inv\Slope Stability\Section B-Circular-PseudoStatic.PLT
PROBLEM DESCRIPTION: Hershfield Res
Section B

BOUNDARY COORDINATES
8 Top Boundaries
9 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	255.00	12.00	255.00	2
2	12.00	255.00	40.00	264.00	2
3	40.00	264.00	58.00	280.00	2
4	58.00	280.00	94.00	314.00	1
5	94.00	314.00	147.00	314.00	1
6	147.00	314.00	194.00	314.00	2
7	194.00	314.00	194.10	324.00	2
8	194.10	324.00	240.00	325.00	2
9	58.00	280.00	147.00	314.00	2

User Specified Y-Origin = 240.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
2 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	125.0	400.0	26.0	0.00	0.0	0
2	125.0	130.0	650.0	28.0	0.00	0.0	0

Specified Peak Ground Acceleration Coefficient (A) = 0.592(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.150(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)
Specified Seismic Pore-Pressure Factor = 0.000
A Critical Failure Surface Searching Method, Using A Random
Technique For Generating Circular Surfaces, Has Been Specified.
3000 Trial Surfaces Have Been Generated.
500 Surface(s) Initiate(s) From Each Of 6 Points Equally Spaced
Along The Ground Surface Between X = 35.00(ft)
and X = 40.00(ft)
Each Surface Terminates Between X = 94.00(ft)
and X = 194.00(ft)
Unless Further Limitations Were Imposed, The Minimum Elevation
At Which A Surface Extends Is Y = 0.00(ft)
7.00(ft) Line Segments Define Each Trial Failure Surface.
Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.
* * Safety Factors Are Calculated By The Modified Bishop Method * *
Total Number of Trial Surfaces Evaluated = 3000
Statistical Data On All Valid FS Values:

FS Max = 2.997 FS Min = 1.278 FS Ave = 2.094
Standard Deviation = 0.497 Coefficient of Variation = 23.72 %
Failure Surface Specified By 14 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	40.00	264.00
2	46.85	265.46
3	53.58	267.37
4	60.17	269.73
5	66.58	272.54
6	72.80	275.76
7	78.77	279.40
8	84.50	283.44
9	89.93	287.85
10	95.06	292.61
11	99.85	297.72
12	104.29	303.13
13	108.35	308.83
14	111.54	314.00

Circle Center At X = 21.90 ; Y = 366.07 ; and Radius = 103.66
Factor of Safety
*** 1.278 ***
Individual data on the 16 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	6.8	1981.5	0.0	0.0	0.	0.	297.2	0.0	0.0
2	6.7	5610.5	0.0	0.0	0.	0.	841.6	0.0	0.0
3	4.4	5454.7	0.0	0.0	0.	0.	818.2	0.0	0.0
4	2.2	3160.4	0.0	0.0	0.	0.	474.1	0.0	0.0
5	6.4	11083.4	0.0	0.0	0.	0.	1662.5	0.0	0.0
6	6.2	12909.8	0.0	0.0	0.	0.	1936.5	0.0	0.0
7	6.0	14061.9	0.0	0.0	0.	0.	2109.3	0.0	0.0
8	5.7	14566.2	0.0	0.0	0.	0.	2184.9	0.0	0.0
9	5.4	14466.1	0.0	0.0	0.	0.	2169.9	0.0	0.0
10	4.1	10973.9	0.0	0.0	0.	0.	1646.1	0.0	0.0
11	1.1	2784.3	0.0	0.0	0.	0.	417.6	0.0	0.0
12	2.3	5482.1	0.0	0.0	0.	0.	822.3	0.0	0.0
13	2.5	5359.8	0.0	0.0	0.	0.	804.0	0.0	0.0
14	4.4	7231.3	0.0	0.0	0.	0.	1084.7	0.0	0.0
15	4.1	3911.1	0.0	0.0	0.	0.	586.7	0.0	0.0
16	3.2	987.7	0.0	0.0	0.	0.	148.2	0.0	0.0

Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	39.00	263.68
2	45.91	264.80
3	52.72	266.40
4	59.41	268.46
5	65.95	270.98
6	72.29	273.94
7	78.41	277.34
8	84.28	281.14
9	89.88	285.35
10	95.18	289.93
11	100.14	294.86
12	104.75	300.12
13	108.99	305.69
14	112.84	311.54
15	114.22	314.00

Circle Center At X = 26.20 ; Y = 364.32 ; and Radius = 101.45
Factor of Safety
*** 1.279 ***
Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	40.00	264.00
2	46.96	264.71
3	53.85	265.95

```
    4         60.63      267.71
    5         67.25      269.98
    6         73.68      272.75
    7         79.88      276.00
    8         85.81      279.71
    9         91.45      283.86
   10         96.75      288.43
   11        101.69      293.39
   12        106.23      298.72
   13        110.36      304.37
   14        114.04      310.32
   15        115.95      314.00
Circle Center At X =    34.18 ; Y =   355.64 ; and Radius =    91.83
       Factor of Safety
      ***    1.280   ***
Failure Surface Specified By 15 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         39.00      263.68
    2         45.92      264.75
    3         52.75      266.28
    4         59.46      268.26
    5         66.04      270.67
    6         72.44      273.50
    7         78.64      276.75
    8         84.61      280.40
    9         90.33      284.44
   10         95.77      288.84
   11        100.91      293.59
   12        105.73      298.67
   13        110.21      304.05
   14        114.32      309.71
   15        117.02      314.00
Circle Center At X =    26.08 ; Y =   369.45 ; and Radius =   106.56
       Factor of Safety
      ***    1.283   ***
Failure Surface Specified By 15 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         39.00      263.68
    2         45.86      265.07
    3         52.61      266.93
    4         59.22      269.23
    5         65.66      271.98
    6         71.90      275.15
    7         77.91      278.74
    8         83.66      282.73
    9         89.13      287.09
   10         94.30      291.82
   11         99.14      296.88
   12        103.62      302.25
   13        107.73      307.92
   14        111.45      313.85
   15        111.53      314.00
Circle Center At X =    21.78 ; Y =   365.93 ; and Radius =   103.69
       Factor of Safety
      ***    1.284   ***
Failure Surface Specified By 15 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         40.00      264.00
    2         46.80      265.67
    3         53.49      267.74
    4         60.04      270.19
    5         66.44      273.02
    6         72.67      276.23
    7         78.69      279.79
    8         84.50      283.69
    9         90.07      287.93
   10         95.39      292.49
```

```
   11        100.42      297.35
   12        105.17      302.50
   13        109.61      307.91
   14        113.72      313.58
   15        113.99      314.00
Circle Center At X =    14.82 ; Y =   381.03 ; and Radius =   119.71
       Factor of Safety
      ***    1.285   ***
Failure Surface Specified By 14 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         39.00      263.68
    2         45.94      264.58
    3         52.78      266.07
    4         59.47      268.14
    5         65.96      270.77
    6         72.20      273.94
    7         78.14      277.63
    8         83.76      281.82
    9         88.99      286.46
   10         93.81      291.54
   11         98.18      297.01
   12        102.07      302.83
   13        105.45      308.96
   14        107.70      314.00
Circle Center At X =    31.94 ; Y =   345.37 ; and Radius =    82.00
       Factor of Safety
      ***    1.285   ***
Failure Surface Specified By 15 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         39.00      263.68
    2         45.84      265.16
    3         52.57      267.08
    4         59.17      269.42
    5         65.61      272.17
    6         71.86      275.32
    7         77.89      278.87
    8         83.69      282.79
    9         89.23      287.06
   10         94.49      291.69
   11         99.45      296.63
   12        104.08      301.88
   13        108.37      307.41
   14        112.30      313.20
   15        112.77      314.00
Circle Center At X =    19.01 ; Y =   372.27 ; and Radius =   110.42
       Factor of Safety
      ***    1.286   ***
Failure Surface Specified By 15 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         40.00      264.00
    2         46.99      264.45
    3         53.91      265.47
    4         60.73      267.05
    5         67.40      269.17
    6         73.88      271.83
    7         80.11      275.01
    8         86.07      278.68
    9         91.71      282.82
   10         97.00      287.41
   11        101.90      292.42
   12        106.37      297.80
   13        110.39      303.53
   14        113.94      309.56
   15        116.08      314.00
Circle Center At X =    37.93 ; Y =   350.12 ; and Radius =    86.15
       Factor of Safety
      ***    1.286   ***
```

Failure Surface Specified By 15 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	38.00	263.36
2	44.89	264.62
3	51.67	266.33
4	58.33	268.50
5	64.83	271.10
6	71.14	274.13
7	77.23	277.58
8	83.08	281.43
9	88.65	285.66
10	93.94	290.25
11	98.90	295.18
12	103.53	300.44
13	107.79	305.99
14	111.67	311.82
15	112.93	314.00

Circle Center At X = 22.63 ; Y = 366.96 ; and Radius = 104.73

Factor of Safety

*** 1.286 ***

**** END OF GSTABL7 OUTPUT ****

APPENDIX B

CALCULATIONS

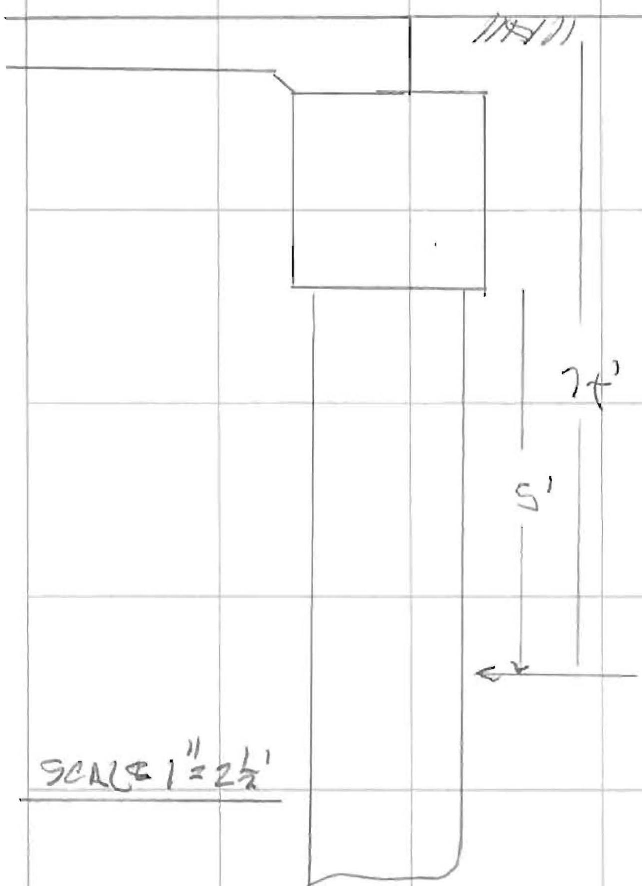
DRILLED PIER AXIAL CAPACITY

SOIL PARAMETERS:

FILL: $\phi = 30^\circ$
$c = 100$ psf
$\gamma = 110$ pcf

CLT: $\phi = 30^\circ$
$c = 600$ psf
$\gamma = 130$ pcf

ASSUME MINIMUM 10' EMBEDMENT INTO Tq
ASSUME GRADE BEAM EMBEDDED 2 + FT INTO Tq



WHEN INCLUDING BASEMENT WALL BACKFILL, DEPTH TO MID POINT IS APPROX 19' FOR USE IN COMPUTING σ_v'

FROM CODUTO (1994)

$$f_s = \beta \sigma_v' + \alpha s_u$$

MID POINT ← 10' DEEP DRILLED PIER

SCALE 1" = 2½'

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$f_s = \beta\, \sigma_v'$ EQ 15.3

WHERE σ_v' = VERTICAL EFFECTIVE STRESS AT MID POINT OF SOIL LAYER

$\sigma_v' = 7 \times 130$ pcf $= 910$ psf

$\beta = 1.5 - 0.135\sqrt{\frac{z}{B_r}}$ EQ 15.5

WHERE $z = 7'$
$B_r = 1$

$\therefore \beta = 1.5 - 0.135\sqrt{7} = 1.14$

$\therefore f_s = 1.14 \times 910 = 1040$ psf

$f_s = \alpha\, S_u$ EQ 15.12

WHERE $S_u = 2c = 1200$ psf

α = FUNCTION OF S_u PER FIG. 15-18

w/ $S_u = 1200$ psf, $\alpha \simeq 0.72$

$\therefore f_s = 0.72 \times 1200 = 864$ psf

$\therefore f_{s\,ULT} = 1040 + 864 = 1904$ psf

FROM TABLE 11.1 w/ GOOD CONTROL, F.S. = 2.0

$\therefore f_{s\,ALL} = 952$ psf – <u>USE 800 psf FOR DESIGN</u>

Most engineers use the α method to compute skin friction resistance in cohesive soils:

$$f_s = \alpha s_u \tag{15.12}$$

Where:

α = adhesion factor

s_u = undrained shear strength of soil along the shaft

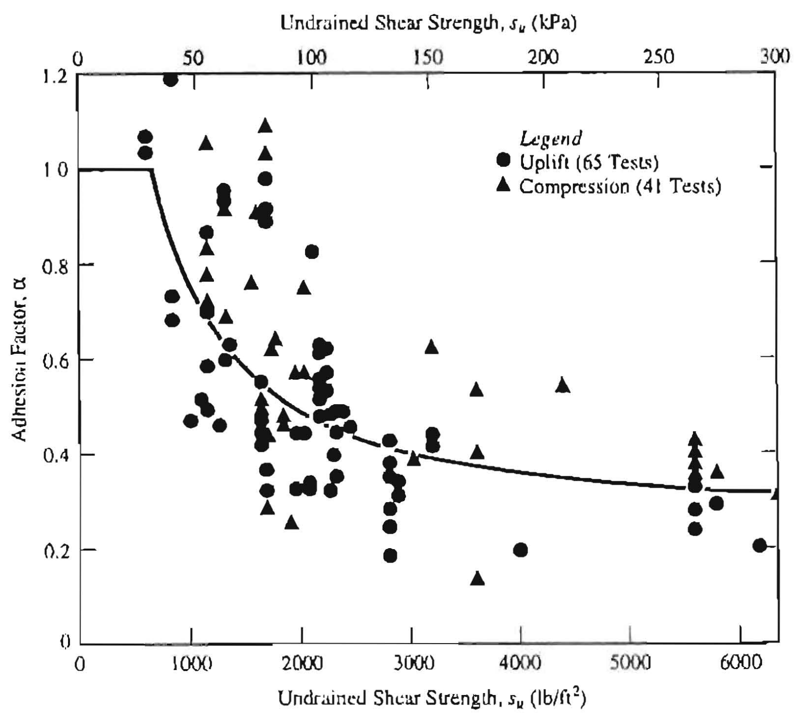

Figure 15.18 α function for drilled shafts (Adapted from Kulhawy and Jackson, 1989; Used with permission of ASCE).

SOURCE: FOUNDATION DESIGN PRINCIPLES & PRACTICES by CODUTO, 1994

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EVALUATE DRAG-DOWN IN THE FILL

FILL ; $\phi = 30°$
$c = 100$ psf
$\gamma = 110$ pcf
$K = 1 - \sin\phi = 0.5$ DUE TO SOFT NATURE

$f_s = k\sigma_v' + \alpha S_u$

WHERE $k = 0.5$
σ_v' VARIES BY ½ OF TOTAL FILL DEPTH
$\alpha = 1.0$, SEE FIG. 15-18
$S_u = 2c = 200$ psf

$f_s = 0.5\sigma_v' + 200$ psf

EVALUATE FILL CONDITIONS ON SITE

FROM FIG 7 OF OUR REPORT, A TOTAL OF 14 PIERS EXIST WESTERLY OF THE LOWER LEVEL CUT/FILL LINE. OR MORE SPECIFICALLY THE CUT/FILL LINE THAT WOULD EXIST BELOW THE BOTTOMS OF THE GRADE BEAMS, IT IS THESE DRILLED PIERS THAT COULD BE IMPACTED BY DRAG-DOWN

THE EXISTING SIDE HILL FILL CAN BE APPROXIMATED AS AN 18° INCLINED PLANE SO IT IS EASY TO CALCULATE THE DEPTH AT EACH DRILLED PIER

THUS, FROM THE CUT/FILL LINE, THE DISTANCE, X TIMES SIN 18° IS THE PIER DEPTH AFFECTED BY DOWN-DRAG

PIER #	DISTANCE FM C/F LINE	PIER DEPTH, ft	σ_v', psf	f_s, psf	DOWN-DRAG LOAD, KIPS	TOP OF PIER RESTRAINT, KIPS
1	29'	9'	715	560	31.7k	36 → 9.5 = 26.5k
2	32'	10'	770	585	36.8k	43.4 → 13.4 = 30k
3	32½'	10'	770	585	36.8k	43.4 - 13.4 = 30k
4	29'	9'	715	560	31.7k	36 → 9.5 = 26.5k
5	30'	10'	770	585	36.8k	43.4 → 6.1 = 37.3k
6	26½'	8'	660	530	26.6k	30 - 4 = 26k
7	24'	8'	660	530	26.6k	30 - 4 = 26k
8	12'	4'	440	420	10.6k	9.5
9	15½'	5'	495	450	14.1k	13.4
10	17'	5'	495	450	14.1k	13.4
11	13'	4'	440	420	10.6k	9.5
12	11'	3'	385	395	7.5k	6.1
13	7'	2'	330	365	4.6k	4
14	5'	2'	330	365	4.6k	4
						322.1k

PIER 1 CALCULATED:

PIER DEPTH $= 29' \sin 18° = 8.96' \sim 9'$

$\sigma_v' = (9'/2 + 2') \times 110$ pcf $= 715$ psf

$f_s = 0.5\,\sigma_v' + 200 = 0.5 \times 715 + 200 = 557.5 \sim 560$ psf

TOP OF PIER RESTRAINT FROM PG. 15

THE LOAD FROM PIER 1 IS REDUCED BY THE LOAD FROM PIER 8

EVALUATE LATERAL LOADING OF DRILLED PIERS

FILL: $\phi = 30°$
$c = 100$ psf
$\gamma = 110$ pcf

$\sigma_H = \sigma_v k_p + 2c\sqrt{k_p}$

WHERE $k_p = \tan^2(45° + \phi/2) = 3.0$

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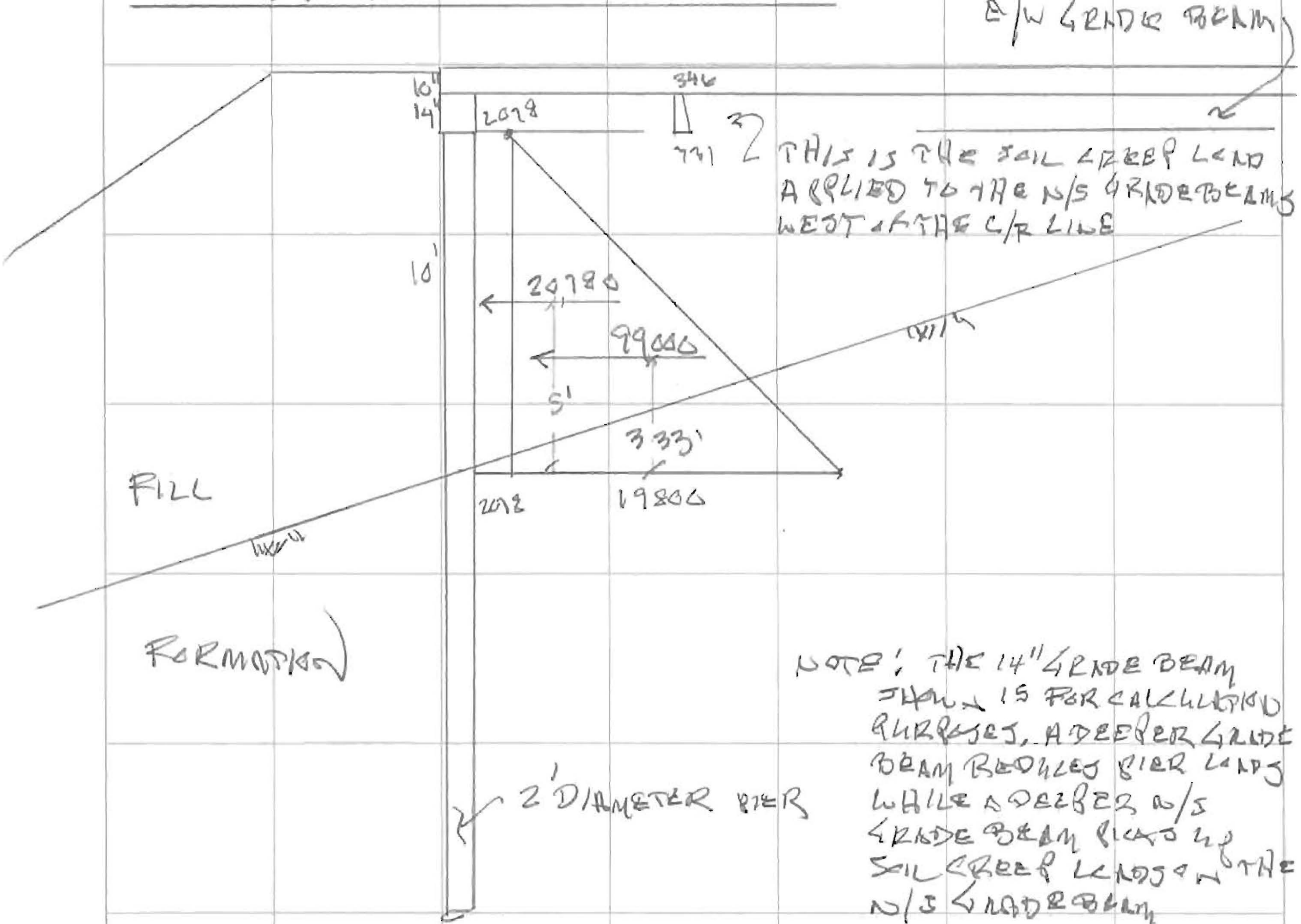
$\sigma_H = Z \gamma K_p + 2c\sqrt{K_p} = Z \times 110 \times 3 + 200\sqrt{3}$

$= 330Z + 346$

FULL SOIL CREEP LOADING = $\sigma_H \times 3\phi$

W/ 2'ϕ PIERS, $\sigma_H = 1980Z + 2078$

EVALUATE 10' DEEP DRILLED PIER



SCALE 1" = 5'

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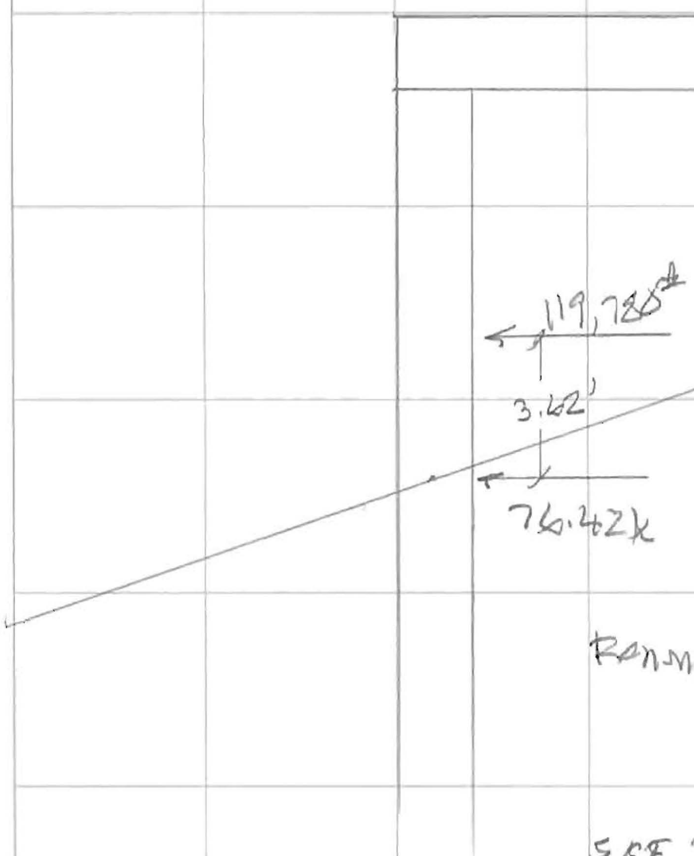
SOLVE FOR LOADS

20780 x 5' = 103 900
99 000 x 3.333' = 329,670

119,780   433,570

L. ARM = 3.62'



→ E/W GRADE BEAM TAKES 43,360#

REQUIRED GRADE BEAM LOAD

$= \dfrac{119780 \times 3.62}{10} = 43,360\#$

REMAINING LATERAL LOAD = 119,780 − 43360

= 76,420#

SEE REESE & MATLOCK - SPREADSHEET (19.8)

ASSUMING A CANTILEVER, $\Delta = 4.87''$

Laterally Loaded Pier Analysis - 8230 Prestwick - 7/9/18
24" Diameter pier w/10 ft cantilever
Reese & Matlock solution - DM7.02

Pile Moment of Inertia, I (in^4):	16,286				
Pile Diameter, D (in):	24.00				
Pile Modulus, E (psi):	3,000,000	Ultimate lateral soil capacity ref: Brom's 1964			
Soil Modulus, f (pci):	60.00	Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2			
Unsupported Cantilevered Height, H (ft):	0.00	Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4			
Depth of Embedment, L (ft):	20.00				
Point of load application, b (ft)	0.00	Soil phi, degrees	35		
		Soil density, pcf	120		
Effective Depth, T (in):	60.56			Pult(kips)	159.48 Long Pile
Effective Depth, T (ft):	5.05			Pult(kips)	177.11 short Pile
Lateral Load, P (kips):	76.42	lever arm	0.00	Note: Use the smaller of the two	
Load Induced Moment, M (Kip-ft):	0.00	Kp	3.69	Also note: to obtain the ultimate capacity for a long pile,	
Embedment Depth Ratio, L/T:	3.96	Myield,Mtotal(Kip-ft):	800	you must balance E15 and L13 to obtain the correct answer	

Computation of Variation in Soil Induced Moment with L/T = 4

Brom's embedment FS = 2.32
FS=0.5*soil-density*D*L^3*Kp/P(L+H) ref. Coduto eq. 17-4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	0.00	0.00	0.00	0
0.25	1.26	0.992	0.240	0.00	92.55	92.55	818
0.50	2.52	0.970	0.467	0.00	180.09	180.09	1592
0.75	3.78	0.926	0.627	0.00	241.80	241.80	2138
1.00	5.05	0.859	0.732	0.00	282.29	282.29	2496
1.25	6.31	0.753	0.767	0.00	295.79	295.79	2615
1.50	7.57	0.640	0.747	0.00	288.07	288.07	2547

Computation of Pile Deformation with L/T = 4

Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"		SLOPE		
0.00	0.00	1.56	2.50	0.00	0.87	0.87	"	0.00986537	Top of Pile Def (in) 0.87 "	
0.25	1.26	1.16	2.07	0.00	0.72	0.72	"	0.00963595		
0.50	2.52	0.82	1.65	0.00	0.57	0.57	"	0.008029956	NOTE: Top of pile deflection is the combination of:	
0.75	3.78	0.52	1.30	0.00	0.45	0.45	"	0.007571101	Ground surface deflection, DEF tot." PLUS	0.87 "
1.00	5.05	0.30	0.97	0.00	0.34	0.34	"	0.006882819	Deflected pile due to angular rotation only, slope*Ht. PLUS	0.00 "
1.25	6.31	0.12	0.67	0.00	0.23	0.23	"	0.005276828	Deflected pile due to loading,Pb^2/6EI(3*L-b)	0.00 "
1.50	7.57	0.03	0.44	0.00	0.15	0.15	"		where: L=lever arm	

PAGE 8 of 12

WITH FULL FIXITY IN THE GRADE BEAM, (CASE II)

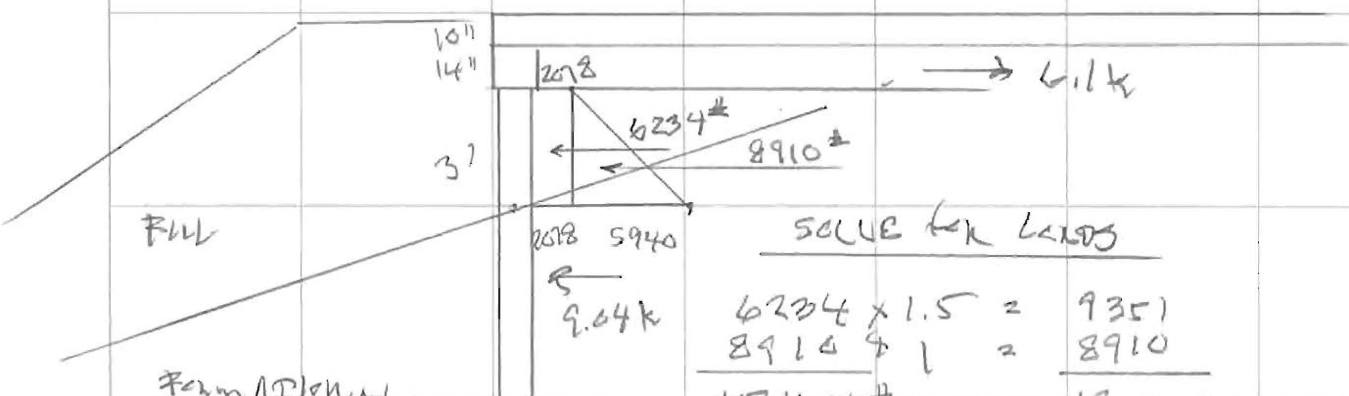
SEE DM-7.2 FIG. 13-5 ATTACHED AT THE
END OF THE CALCULATION PACKAGE

$$\Delta = \frac{F_\delta P T^3}{EI} = \frac{1.03 \times 76,420 \times 66.54^3}{3 \times 10^6 \times 16286} = 0.36''$$

SO THE ACTUAL DOWNSLOPE DEFLECTION OF THE
DRILLED PIER AT THE CONTACT IS ABOUT $\frac{1}{2}''$

SO OK

② EVALUATE 3' DEEP DRILLED PIER



SOLVE FOR LOADS

6234 × 1.5	=	9351
8910 × 1	=	8910
15144#		18,261 ft-#

L. ARM = 1.21

REQUIRED GRADE BEAM LOAD = 6,102#

FORMATIONAL LATERAL LOAD = 9036#

SEE RESULTS MATLAB SPREADSHEET (09.10)

Laterally Loaded Pier Analysis - 8230 Prestwick - 7/9/18
24" Diameter pier w/3 ft cantilever
Reese & Matlock solution - DM7.02

Pile Moment of Inertia, I (in^4):	16,286				
Pile Diameter, D (in):	24.00				
Pile Modulus, E (psi):	3,000,000	Ultimate lateral soil capacity ref: Brom's 1964			
Soil Modulus, f (pci):	60.00	Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2			
Unsupported Cantilevered Height, H (ft):	0.00	Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4			
Depth of Embedment, L (ft):	15.00				
Point of load application, b (ft)	0.00	Soil phi, degrees	35		
		Soil density, pcf	120		
Effective Depth, T (in):	60.56			Pult(kips)	463.68 Long Pile
Effective Depth, T (ft):	5.05			Pult(kips)	99.62 short Pile
Lateral Load, P (kips):	9.04	lever arm	0.00	Note: Use the smaller of the two	
Load Induced Moment, M (Kip-ft):	0.00	Kp	3.69	Also note: to obtain the ultimate capacity for a long pile,	
Embedment Depth Ratio, L/T:	2.97	Myield,Mtotal(Kip-ft):	800	you must balance E15 and L13 to obtain the correct answer	

Computation of Variation in Soil Induced Moment with L/T = 4

Brom's embedment FS = 11.02
FS=0.5*soil-density*D*L^3*Kp/P(L+H) ref. Coduto eq. 17-4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	0.00	0.00	0.00	0
0.25	1.26	0.992	0.240	0.00	10.95	10.95	97
0.50	2.52	0.970	0.467	0.00	21.30	21.30	188
0.75	3.78	0.926	0.627	0.00	28.60	28.60	253
1.00	5.05	0.859	0.732	0.00	33.39	33.39	295
1.25	6.31	0.753	0.767	0.00	34.99	34.99	309
1.50	7.57	0.640	0.747	0.00	34.08	34.08	301

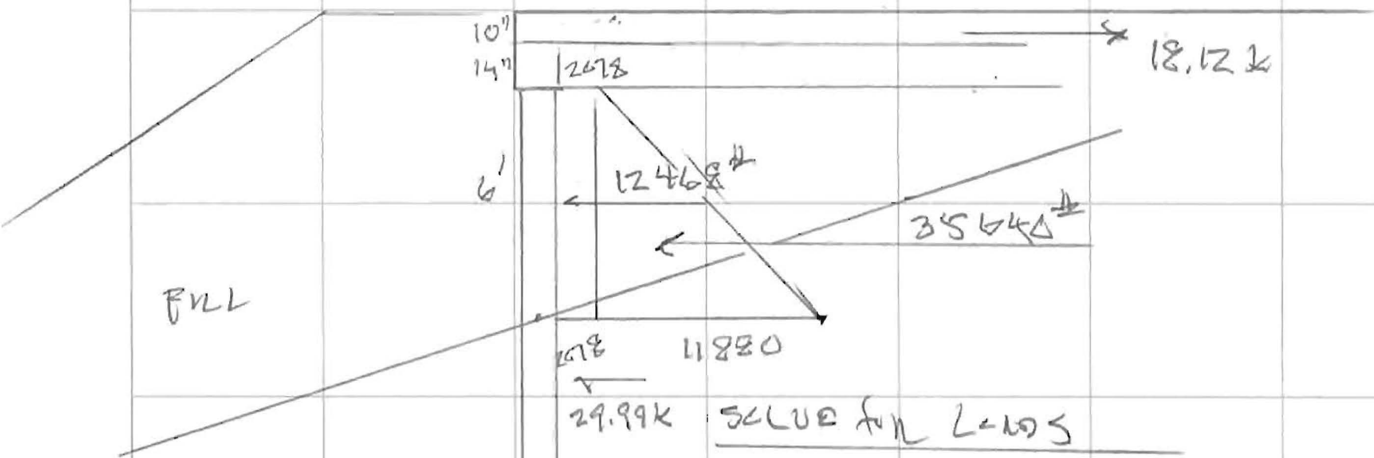
Computation of Pile Deformation with L/T = 4

Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"	SLOPE
0.00	0.00	1.56	2.50	0.00	0.10	0.10 "	0.00116701
0.25	1.26	1.16	2.07	0.00	0.09	0.09 "	0.00113987
0.50	2.52	0.82	1.65	0.00	0.07	0.07 "	0.000949893
0.75	3.78	0.52	1.30	0.00	0.05	0.05 "	0.000895613
1.00	5.05	0.30	0.97	0.00	0.04	0.04 "	0.000814194
1.25	6.31	0.12	0.67	0.00	0.03	0.03 "	0.000624215
1.50	7.57	0.03	0.44	0.00	0.02	0.02 "	

Top of Pile Def (in) 0.10 "

NOTE: Top of pile deflection is the combination of:	
Ground surface deflection, DEF tot." PLUS	0.10 "
Deflected pile due to angular rotation only, slope*Ht. PLUS	0.00 "
Deflected pile due to loading, Pb^2/6EI(3*L-b)	0.00 "
where: L=lever arm	

EVALUATE 6' DEEP DRILLED PILE



FORMATION

SOLVE FOR LOADS

12468 × 3 = 37,404
35,640 × 2 = 71,280
48,108# 108,684 ft-#

L. ARM = 2.26'

REQUIRED GRADE BEAM LOAD = $\frac{48,108 \times 2.26}{6}$ = 18,121

FORMATIONAL LATERAL LOAD = 29,987#

SEE ~~REBAR~~ MAT'L SPREAD SHEET (PG. 12)

Laterally Loaded Pier Analysis - 8230 Prestwick - 7/9/18
24" Diameter pier w/6 ft cantilever
Reese & Matlock solution - DM7.02

Pile Moment of Inertia, I (in^4):	16,286
Pile Diameter, D (in):	24.00
Pile Modulus, E (psi):	3,000,000
Soil Modulus, f (pci):	60.00
Unsupported Cantilevered Height, H (ft):	0.00
Depth of Embedment, L (ft):	15.00
Point of load application, b (ft)	0.00
Effective Depth, T (in):	60.56
Effective Depth, T (ft):	5.05
Lateral Load, P (kips):	29.99
Load Induced Moment, M (Kip-ft):	0.00
Embedment Depth Ratio, L/T:	2.97

Ultimate lateral soil capacity ref: Brom's 1964
Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2
Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4

Soil phi, degrees	35
Soil density, pcf	120
lever arm	0.00
Kp	3.69
Myield,Mtotal(Kip-ft):	800

Pult(kips)	254.57	Long Pile
Pult(kips)	99.62	short Pile

Note: Use the smaller of the two
Also note: to obtain the ultimate capacity for a long pile, you must balance E15 and L13 to obtain the correct answer

Computation of Variation in Soil Induced Moment with L/T = 4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	0.00	0.00	0.00	0
0.25	1.26	0.992	0.240	0.00	36.32	36.32	321
0.50	2.52	0.970	0.467	0.00	70.68	70.68	625
0.75	3.78	0.926	0.627	0.00	94.89	94.89	839
1.00	5.05	0.859	0.732	0.00	110.78	110.78	980
1.25	6.31	0.753	0.767	0.00	116.08	116.08	1026
1.50	7.57	0.640	0.747	0.00	113.05	113.05	1000

Brom's embedment FS = 3.32
FS=0.5*soil-density*D*L^3*Kp/P(L+H) ref. Coduto eq. 17-4

Computation of Pile Deformation with L/T = 4

Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"	SLOPE
0.00	0.00	1.56	2.50	0.00	0.34	0.34 "	0.00387153
0.25	1.26	1.16	2.07	0.00	0.28	0.28 "	0.00378150
0.50	2.52	0.82	1.65	0.00	0.22	0.22 "	0.003151248
0.75	3.78	0.52	1.30	0.00	0.18	0.18 "	0.002971177
1.00	5.05	0.30	0.97	0.00	0.13	0.13 "	0.00270107
1.25	6.31	0.12	0.67	0.00	0.09	0.09 "	0.00207082
1.50	7.57	0.03	0.44	0.00	0.06	0.06 "	

Top of Pile Def (in) 0.34 "

NOTE: Top of pile deflection is the combination of:	
Ground surface deflection, DEF tot" PLUS	0.34 "
Deflected pile due to angular rotation only, slope*Ht. PLUS	0.00 "
Deflected pile due to loading,Pb^2/6EI(3*L-b)	0.00 "
where: L=lever arm	

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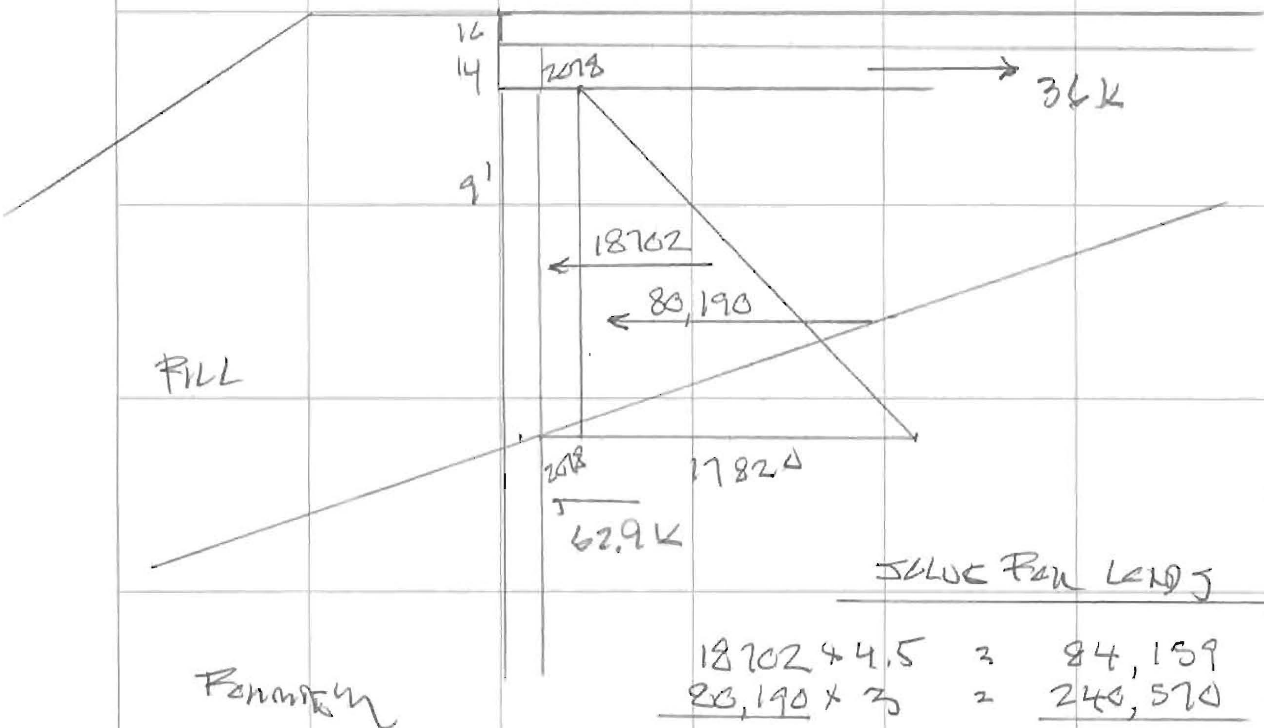
EVALUATE 9' DEEP DRILLED PIER



SLIDE PLANE LOADS

18702 × 4.5	=	84,159
80,190 × 3	=	240,570
98,892		324,729

L. ARM = 3.28'

REQUIRED GRADE BEAM LOAD = $\frac{98,892 \times 3.28}{9}$ = 36,041

FOUNDATION LATERAL LOAD = 62,851

SEE REESE & MATLOCK SPREADSHEET (pg. 14)

Laterally Loaded Pier Analysis - 8230 Prestwick - 7/9/18
24" Diameter pier w/9 ft cantilever
Reese & Matlock solution - DM7.02

Pile Moment of Inertia, I (in^4):	16,286				
Pile Diameter, D (in):	24.00				
Pile Modulus, E (psi):	3,000,000	Ultimate lateral soil capacity ref: Brom's 1964			
Soil Modulus, f (pci):	60.00	Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2			
Unsupported Cantilevered Height, H (ft):	0.00	Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4			
Depth of Embedment, L (ft):	20.00				
Point of load application, b (ft)	0.00	Soil phi, degrees	35		
		Soil density, pcf	120		
Effective Depth, T (in):	60.56			Pult(kips)	175.78 Long Pile
Effective Depth, T (ft):	5.05			Pult(kips)	177.11 short Pile
Lateral Load, P (kips):	62.90	lever arm	0.00	Note: Use the smaller of the two	
Load Induced Moment, M (Kip-ft):	0.00	Kp	3.69	Also note: to obtain the ultimate capacity for a long pile,	
Embedment Depth Ratio, L/T:	3.96	Myield,Mtotal(Kip-ft):	800	you must balance E15 and L13 to obtain the correct answer	

Computation of Variation in Soil Induced Moment with L/T = 4

Brom's embedment FS = 2.82
FS=0.5*soil-density*D*L^3*Kp/P(L+H) ref. Coduto eq. 17-4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	0.00	0.00	0.00	0
0.25	1.26	0.992	0.240	0.00	76.18	76.18	674
0.50	2.52	0.970	0.467	0.00	148.23	148.23	1311
0.75	3.78	0.926	0.627	0.00	199.02	199.02	1760
1.00	5.05	0.859	0.732	0.00	232.35	232.35	2054
1.25	6.31	0.753	0.767	0.00	243.46	243.46	2153
1.50	7.57	0.640	0.747	0.00	237.11	237.11	2097

Computation of Pile Deformation with L/T = 4

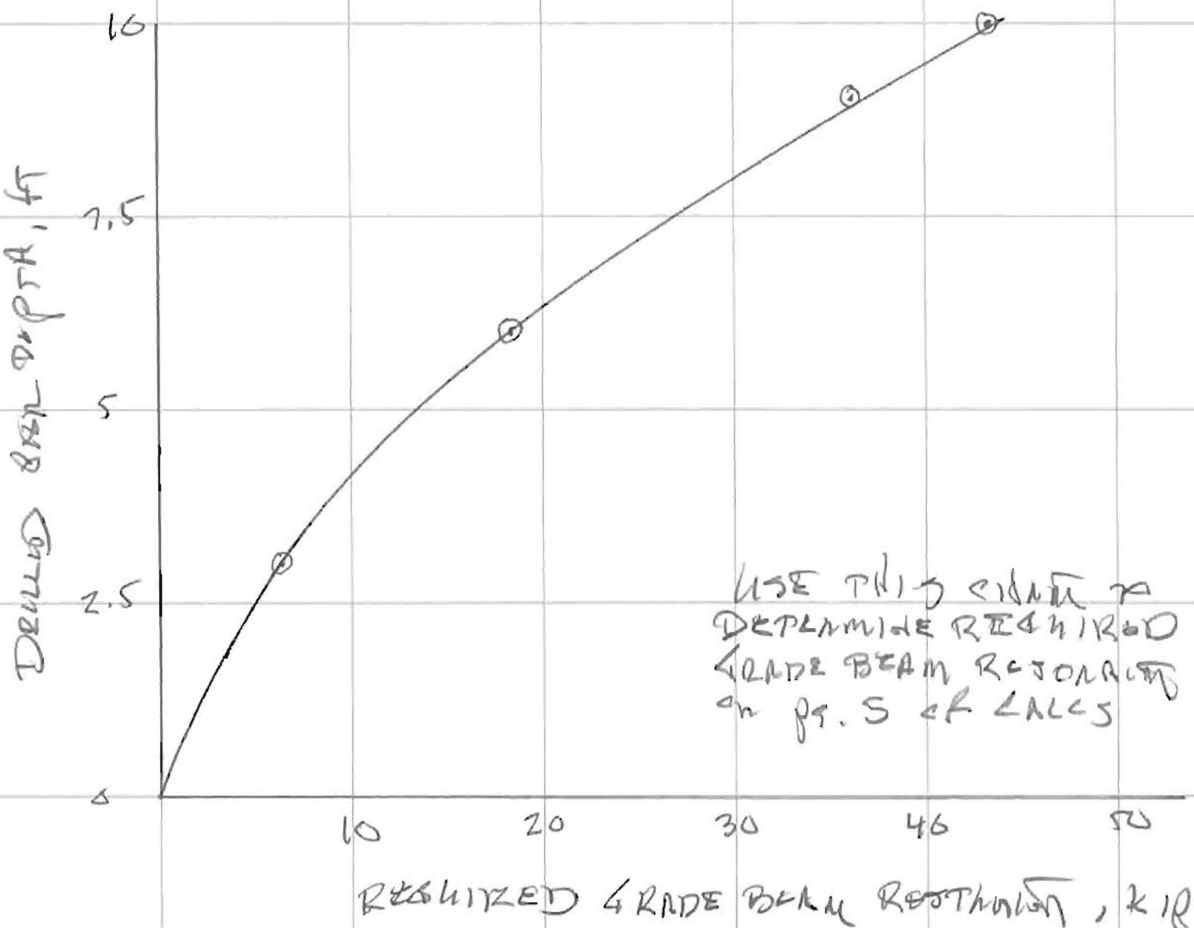
Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"	SLOPE
0.00	0.00	1.56	2.50	0.00	0.71	0.71 "	0.00812002
0.25	1.26	1.16	2.07	0.00	0.59	0.59 "	0.00793118
0.50	2.52	0.82	1.65	0.00	0.47	0.47 "	0.00660932
0.75	3.78	0.52	1.30	0.00	0.37	0.37 "	0.006231644
1.00	5.05	0.30	0.97	0.00	0.28	0.28 "	0.005665131
1.25	6.31	0.12	0.67	0.00	0.19	0.19 "	0.004343267
1.50	7.57	0.03	0.44	0.00	0.13	0.13 "	

Top of Pile Def (in) 0.71 "

NOTE: Top of pile deflection is the combination of:

Ground surface deflection, DEF tot." PLUS	0.71 "
Deflected pile due to angular rotation only, slope*Ht. PLUS	0.00 "
Deflected pile due to loading,Pb^2/6EI(3*L-b)	0.00 "
where: L=lever arm	

EVALUATE EAST/WEST GRADE BEAM LOADS



USE THIS CHART TO DETERMINE REQUIRED GRADE BEAM RESTRAINT LISTED ON PG. 5 OF CALCS

AS A TEST, COMPARE TERZAGHI'S RECOMMENDED ACTIVE WALL PRESSURES ACROSS THE PROPERTY ASSUMING THAT WE WERE TO RESTRAIN THE ENTIRE WESTERN SLOPE.

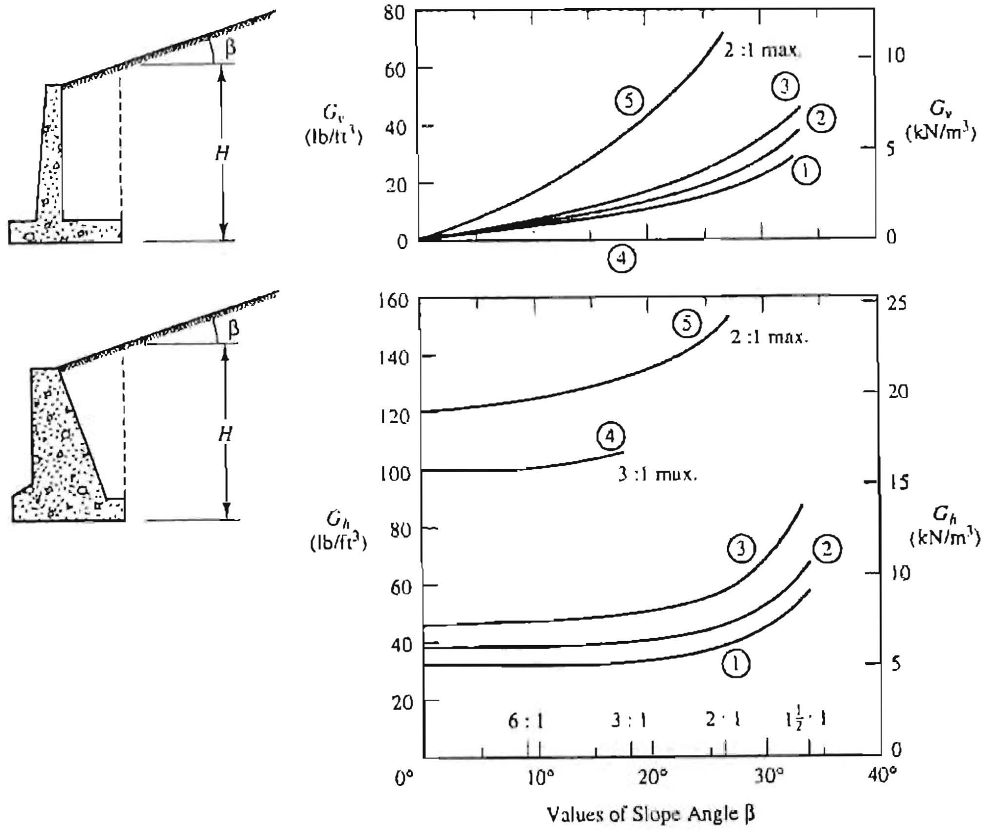


Figure 23.13 Charts for estimating the loads acting against a retaining wall beneath a planar ground surface (Adapted from Terzaghi and Peck, 1967).

This method probably is appropriate only for walls less than about 20 ft (6 m) in height.

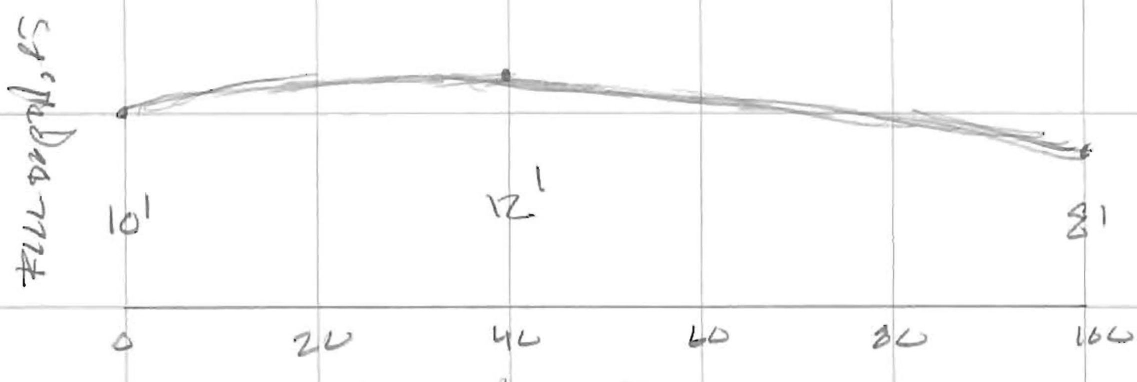
TABLE 23.3 CLASSIFICATION OF SOIL TYPES

Soil Type	Description
1	Coarse grained soil without admixture of fine soil particles, very permeable (i.e., clean sand or gravel).
2	Coarse-grained soil of low permeability due to admixture of particles of silt size.
3	Residual soil with stones, fine silty sand, and granular materials with conspicuous clay content.
4	Very soft clay, organic silts, or silty clays.
5	Medium or stiff clay, deposited in chunks and protected in such a way that a negligible amount of water enters the spaces between the chunks during floods or heavy rains. If this condition cannot be satisfied, the clay should not be used as backfill material. With increasing stiffness of the clay, danger to the wall due to infiltration of water increases rapidly.

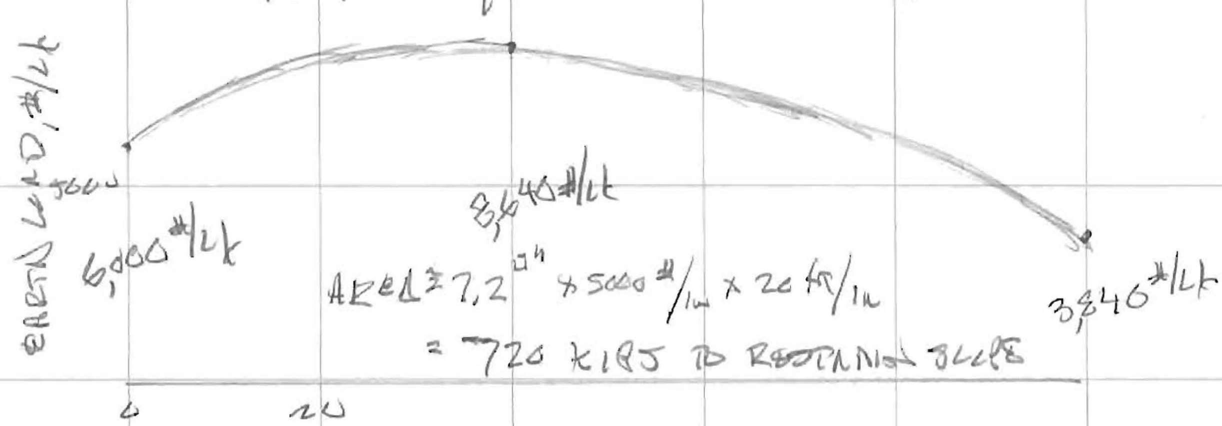
Adapted from Terzaghi and Peck (1967).

LOT WIDTH = 100'

ACROSS THE LOT, FILL DEPTH FROM THE LOWER LEVEL IS APPROX AS FOLLOWS



ASSUMING A TYPE 5 SOIL, $F = \frac{1}{2} \cdot 120 \cdot H^2$

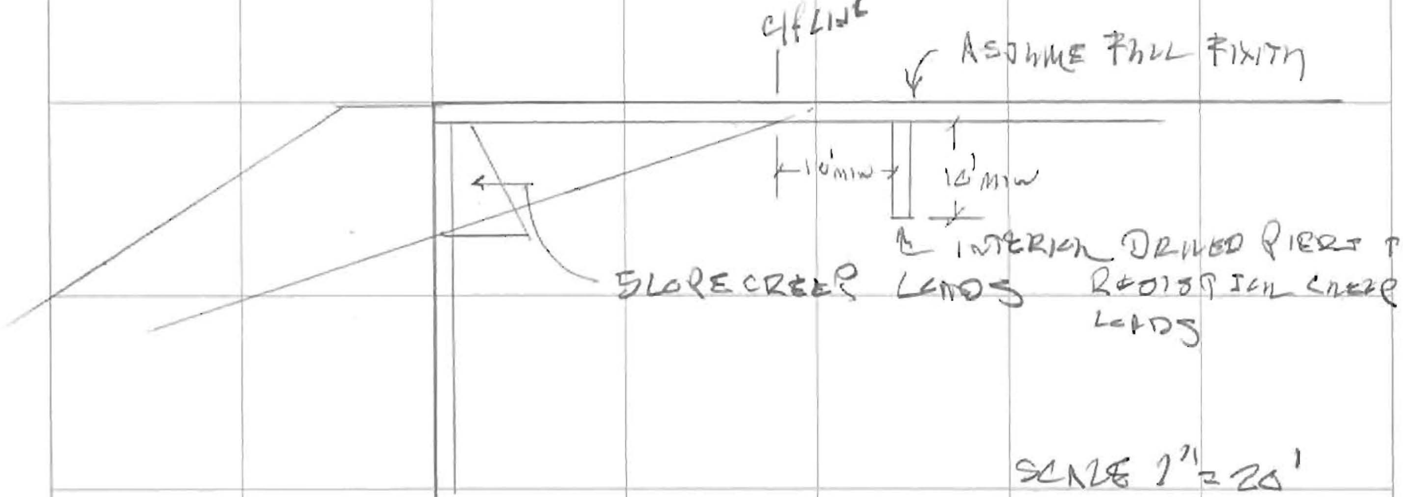


ACTUAL PROPOSED RESTRAINT IN GRADE BEAMS = 322.1 KIPS

TOTAL RESTRAINT INCLUDING LATERAL RESISTANCE IN BEDROCK ≈ 875 KIPS

WHICH IS CLOSE TO TERRACOSTA'S RECOMMENDATION

EVALUATE AVAILABLE RESTRAINT PROVIDED BY THE PIERS BATTERY + THE C/F LINE



SCALE 1" = 20'

SEE DM-7.2 CASE II LOADING CONDITION AT END OF CALCULATIONS PACKAGE ALONG W/ SPREAD SHEET ON PG. 19

FOR CASE I, PG. 19 $\Delta = 4.25"$ ASSUMING P = 30 KIPS

PER CASE II, $\Delta = \frac{F_\delta P T^3}{EI} = 0.11"$ ≈ VERY REASONABLE

WHERE: $F_\delta = 1.1$ FROM FIG. 13.5 DM-7.2
$P = 30$ K FOR ILLUSTRATION CALC
$T = 54.67"$ FROM PG. 19
$E = 3 \times 10^6$ "
$I = 16,286$ IN^4 "

THE STRUCTURAL ENGINEER CAN EASILY RECALCULATE Δ FOR CASE II PER THE ATTACHED DM-7.2 FOR ANY LOADING CONDITION

Laterally Loaded Pier Analysis - 8230 Prestwick - 7/9/18

24" Diameter pier in cut

Reese & Matlock solution - DM7.02

Pile Moment of Inertia, I (in^4):	16,286					
Pile Diameter, D (in):	24.00					
Pile Modulus, E (psi):	3,000,000	Ultimate lateral soil capacity ref: Brom's 1964				
Soil Modulus, f (pci):	100.00	Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2				
Unsupported Cantilevered Height, H (ft):	0.00	Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4				
Depth of Embedment, L (ft):	10.00					
Point of load application, b (ft)	0.00	Soil phi, degrees	35			
		Soil density, pcf	120			
Effective Depth, T (in):	54.67			Pult(kips)	254.53	Long Pile
Effective Depth, T (ft):	4.56			Pult(kips)	44.28	short Pile
Lateral Load, P (kips):	30.00	lever arm	0.00	Note: Use the smaller of the two		
Load Induced Moment, M (Kip-ft):	0.00	Kp	3.69	Also note: to obtain the ultimate capacity for a long pile,		
Embedment Depth Ratio, L/T:	2.19	Myield,Mtotal(Kip-ft):	800	you must balance E15 and L13 to obtain the correct answer		

Computation of Variation in Soil Induced Moment with L/T = 4

Brom's embedment FS = 1.48

FS=0.5*soil-density*D*L^3*Kp/P(L+H) ref. Coduto eq. 17-4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	0.00	0.00	0.00	0
0.25	1.14	0.992	0.240	0.00	32.80	32.80	290
0.50	2.28	0.970	0.467	0.00	63.83	63.83	564
0.75	3.42	0.926	0.627	0.00	85.70	85.70	758
1.00	4.56	0.859	0.732	0.00	100.05	100.05	885
1.25	5.70	0.753	0.767	0.00	104.84	104.84	927
1.50	6.83	0.640	0.747	0.00	102.11	102.11	903

Computation of Pile Deformation with L/T = 4

Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"	SLOPE
0.00	0.00	1.56	2.50	0.00	0.25	0.25 "	0.00315710
0.25	1.14	1.16	2.07	0.00	0.21	0.21 "	0.00308368
0.50	2.28	0.82	1.65	0.00	0.17	0.17 "	0.002569732
0.75	3.42	0.52	1.30	0.00	0.13	0.13 "	0.00242289
1.00	4.56	0.30	0.97	0.00	0.10	0.10 "	0.002202628
1.25	5.70	0.12	0.67	0.00	0.07	0.07 "	0.001688681
1.50	6.83	0.03	0.44	0.00	0.04	0.04 "	

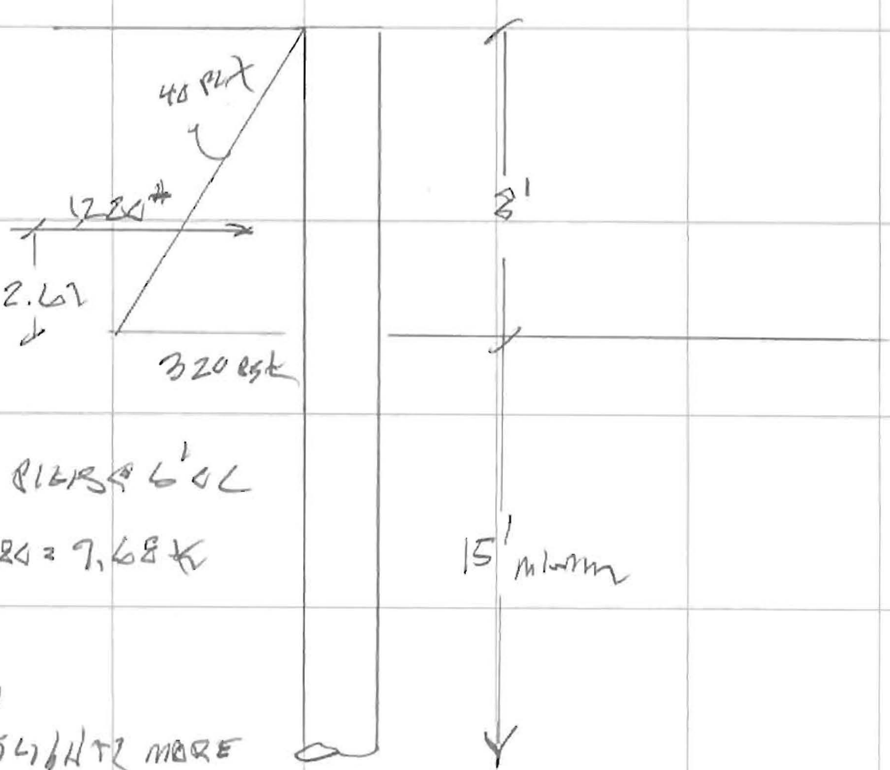
Top of Pile Def (in)

0.25 "

NOTE: Top of pile deflection is the combination of:	
Ground surface deflection, DEF tot." PLUS	0.25 "
Deflected pile due to angular rotation only, slope*Ht. PLUS	0.00 "
Deflected pile due to loading,Pb^2/6EI(3*L-b)	0.00 "
where: L=lever arm	

ALTERNATE 2' ϕ DRILLED PIER WALL for PROPERTY LINE WALL:

$H = 8'$
EFP = 40 pcf



W/ DRILLED PIERS @ 6' o.c.

$P = 6 \times 1280 = 7.68\ k$

@ $\Delta \approx 0.22''$
actually slightly more

~ 1/4" w/ L/T = 3.07 assuming 15' embedment

see spreadsheet ~ pg. 21

Laterally Loaded Pier Analysis - 8230 Prestwick - 7/9/18

24" Diameter property line drilled pier wall w/8 ft cantilever --> Assuming 6 ft OC

Reese & Matlock solution - DM7.02

Pile Moment of Inertia, I (in^4):	16,286				
Pile Diameter, D (in):	24.00				
Pile Modulus, E (psi):	3,000,000	Ultimate lateral soil capacity ref: Brom's 1964			
Soil Modulus, f (pci):	100.00	Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2			
Unsupported Cantilevered Height, H (ft):	8.00	Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4			
Depth of Embedment, L (ft):	14.00				
Point of load application, b (ft)	2.67	Soil phi, degrees	35		
		Soil density, pcf	120		
Effective Depth, T (in):	54.67			Pult(kips)	83.42 Long Pile
Effective Depth, T (ft):	4.56			Pult(kips)	55.23 short Pile
Lateral Load, P (kips):	7.68	lever arm	2.67	Note: Use the smaller of the two	
Load Induced Moment, M (Kip-ft):	20.51	Kp	3.69	Also note: to obtain the ultimate capacity for a long pile,	
Embedment Depth Ratio, L/T:	3.07	Myield,Mtotal(Kip-ft);	800	you must balance E15 and L13 to obtain the correct answer	

Computation of Variation in Soil Induced Moment with L/T = 4

Brom's embedment FS = 7.19

FS=0.5*soil-density*D*L^3*Kp/P(L+H) ref. Coduto eq. 17-4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	20.51	0.00	20.51	181
0.25	1.14	0.992	0.240	20.34	8.40	28.74	254
0.50	2.28	0.970	0.467	19.89	16.34	36.23	320
0.75	3.42	0.926	0.627	18.99	21.94	40.93	362
1.00	4.56	0.859	0.732	17.61	25.61	43.23	382
1.25	5.70	0.753	0.767	15.44	26.84	42.28	374
1.50	6.83	0.640	0.747	13.12	26.14	39.26	347

Computation of Pile Deformation with L/T = 4

Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"	SLOPE
0.00	0.00	1.56	2.50	0.02	0.06	0.09 "	0.00125902
0.25	1.14	1.16	2.07	0.02	0.05	0.07 "	0.00118079
0.50	2.28	0.82	1.65	0.01	0.04	0.05 "	0.001003575
0.75	3.42	0.52	1.30	0.01	0.03	0.04 "	0.000866787
1.00	4.56	0.30	0.97	0.00	0.02	0.03 "	0.000748189
1.25	5.70	0.12	0.67	0.00	0.02	0.02 "	0.000510682
1.50	6.83	0.03	0.44	0.00	0.01	0.01 "	

Top of Pile Def (in) 0.22 "

NOTE: Top of pile deflection is the combination of:	
Ground surface deflection, DEF tot." PLUS	0.09 "
Deflected pile due to angular rotation only, slope*Ht. PLUS	0.12 "
Deflected pile due to loading,Pb^2/6EI(3*L-b)	0.01 "
where: L=lever arm	

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City of San Diego
Development Services
1222 First Ave., MS-302
San Diego, CA 92101
(619) 446-5000

Storm Water Requirements Applicability Checklist

FORM **DS-560**
OCTOBER 2016

Project Address: 8230 PRESTWICK DRIVE	Project Number *(for City Use Only)*:

SECTION 1. Construction Storm Water BMP Requirements:

All construction sites are required to implement construction BMPs in accordance with the performance standards in the Storm Water Standards Manual. Some sites are additionally required to obtain coverage under the State Construction General Permit (CGP)[1], which is administered by the State Water Resources Control Board.

For all projects complete PART A: If project is required to submit a SWPPP or WPCP, continue to PART B.

PART A: Determine Construction Phase Storm Water Requirements.

1. Is the project subject to California's statewide General NPDES permit for Storm Water Discharges Associated with Construction Activities, also known as the State Construction General Permit (CGP)? (Typically projects with land disturbance greater than or equal to 1 acre.)

☐ Yes; SWPPP required, skip questions 2-4 ☒ No; next question

2. Does the project propose construction or demolition activity, including but not limited to, clearing, grading, grubbing, excavation, or any other activity resulting in ground disturbance and contact with storm water runoff?

☒ Yes; WPCP required, skip 3-4 ☐ No; next question

3. Does the project propose routine maintenance to maintain original line and grade, hydraulic capacity, or original purpose of the facility? (Projects such as pipeline/utility replacement)

☐ Yes; WPCP required, skip 4 ☐ No; next question

4. Does the project only include the following Permit types listed below?

- Electrical Permit, Fire Alarm Permit, Fire Sprinkler Permit, Plumbing Permit, Sign Permit, Mechanical Permit, Spa Permit.
- Individual Right of Way Permits that exclusively include only ONE of the following activities: water service, sewer lateral, or utility service.
- Right of Way Permits with a project footprint less than 150 linear feet that exclusively include only ONE of the following activities: curb ramp, sidewalk and driveway apron replacement, pot holing, curb and gutter replacement, and retaining wall encroachments.

☐ Yes; no document required

Check one of the boxes below, and continue to PART B:

☐ If you checked "Yes" for question 1, **a SWPPP is REQUIRED. Continue to PART B**

☒ If you checked "No" for question 1, and checked "Yes" for question 2 or 3, **a WPCP is REQUIRED.** If the project proposes less than 5,000 square feet of ground disturbance AND has less than a 5-foot elevation change over the entire project area, a Minor WPCP may be required instead. **Continue to PART B.**

☐ If you checked "No" for all questions 1-3, and checked "Yes" for question 4 PART B **does not apply and no document is required. Continue to Section 2.**

1. More information on the City's construction BMP requirements as well as CGP requirements can be found at: www.sandiego.gov/stormwater/regulations/index.shtml

PART B: Determine Construction Site Priority

This prioritization must be completed within this form, noted on the plans, and included in the SWPPP or WPCP. The city reserves the right to adjust the priority of projects both before and after construction. Construction projects are assigned an inspection frequency based on if the project has a "high threat to water quality." The City has aligned the local definition of "high threat to water quality" to the risk determination approach of the State Construction General Permit (CGP). The CGP determines risk level based on project specific sediment risk and receiving water risk. Additional inspection is required for projects within the Areas of Special Biological Significance (ASBS) watershed. **NOTE:** The construction priority does **NOT** change construction BMP requirements that apply to projects; rather, it determines the frequency of inspections that will be conducted by city staff.

Complete PART B and continued to Section 2

1. ☒ **ASBS**

   a. Projects located in the ASBS watershed.

2. ☐ **High Priority**

   a. Projects 1 acre or more determined to be Risk Level 2 or Risk Level 3 per the Construction General Permit and not located in the ASBS watershed.

   b. Projects 1 acre or more determined to be LUP Type 2 or LUP Type 3 per the Construction General Permit and not located in the ASBS watershed.

3. ☐ **Medium Priority**

   a. Projects 1 acre or more but not subject to an ASBS or high priority designation.

   b. Projects determined to be Risk Level 1 or LUP Type 1 per the Construction General Permit and not located in the ASBS watershed.

4. ☐ **Low Priority**

   a. Projects requiring a Water Pollution Control Plan but not subject to ASBS, high, or medium priority designation.

SECTION 2. Permanent Storm Water BMP Requirements.

Additional information for determining the requirements is found in the Storm Water Standards Manual.

PART C: Determine if Not Subject to Permanent Storm Water Requirements.

Projects that are considered maintenance, or otherwise not categorized as "new development projects" or "redevelopment projects" according to the Storm Water Standards Manual are not subject to Permanent Storm Water BMPs.

If "yes" is checked for any number in Part C, proceed to Part F and check "Not Subject to Permanent Storm Water BMP Requirements".

If "no" is checked for all of the numbers in Part C continue to Part D.

1.	Does the project only include interior remodels and/or is the project entirely within an existing enclosed structure and does not have the potential to contact storm water?	☐ Yes ☒ No
2.	Does the project only include the construction of overhead or underground utilities without creating new impervious surfaces?	☐ Yes ☒ No
3.	Does the project fall under routine maintenance? Examples include, but are not limited to: roof or exterior structure surface replacement, resurfacing or reconfiguring surface parking lots or existing roadways without expanding the impervious footprint, and routine replacement of damaged pavement (grinding, overlay, and pothole repair).	☐ Yes ☒ No

PART D: PDP Exempt Requirements.

PDP Exempt projects are required to implement site design and source control BMPs.

If "yes" was checked for any questions in Part D, continue to Part F and check the box labeled "PDP Exempt."

If "no" was checked for all questions in Part D, continue to Part E.

1. **Does the project ONLY include new or retrofit sidewalks, bicycle lanes, or trails that:**
   - **Are designed and constructed to direct storm water runoff to adjacent vegetated areas, or other non-erodible permeable areas? Or;**
   - **Are designed and constructed to be hydraulically disconnected from paved streets and roads? Or;**
   - **Are designed and constructed with permeable pavements or surfaces in accordance with the Green Streets guidance in the City's Storm Water Standards manual?**

   ☐ Yes; PDP exempt requirements apply ☒ No; next question

2. Does the project ONLY include retrofitting or redeveloping existing paved alleys, streets or roads designed and constructed in accordance with the Green Streets guidance in the City's Storm Water Standards Manual?

   ☐ Yes; PDP exempt requirements apply ☒ No; project not exempt.

PART E: Determine if Project is a Priority Development Project (PDP).

Projects that match one of the definitions below are subject to additional requirements including preparation of a Storm Water Quality Management Plan (SWQMP).

If "yes" is checked for any number in PART E, continue to PART F and check the box labeled "Priority Development Project".

If "no" is checked for every number in PART E, continue to PART F and check the box labeled "Standard Development Project".

1. **New Development that creates 10,000 square feet or more of impervious surfaces collectively over the project site.** This includes commercial, industrial, residential, mixed-use, and public development projects on public or private land. ☐ Yes ☒ No
2. **Redevelopment project that creates and/or replaces 5,000 square feet or more of impervious surfaces on an existing site of 10,000 square feet or more of impervious surfaces**. This includes commercial, industrial, residential, mixed-use, and public development projects on public or private land. ☐ Yes ☒ No
3. **New development or redevelopment of a restaurant.** Facilities that sell prepared foods and drinks for consumption, including stationary lunch counters and refreshment stands selling prepared foods and drinks for immediate consumption (SIC 5812), and where the land development creates and/or replace 5,000 square feet or more of impervious surface. ☐ Yes ☒ No
4. **New development or redevelopment on a hillside.** The project creates and/or replaces 5,000 square feet or more of impervious surface (collectively over the project site) and where the development will grade on any natural slope that is twenty-five percent or greater. ☐ Yes ☒ No
5. **New development or redevelopment of a parking lot that creates and/or replaces 5,000 square feet or more of impervious surface (collectively over the project site).** ☐ Yes ☒ No
6. **New development or redevelopment of streets, roads, highways, freeways, and driveways.** The project creates and/or replaces 5,000 square feet or more of impervious surface (collectively over the project site). ☐ Yes ☒ No

7. **New development or redevelopment discharging directly to an Environmentally Sensitive Area.** The project creates and/or replaces 2,500 square feet of impervious surface (collectively over project site), and discharges directly to an Environmentally Sensitive Area (ESA). "Discharging directly to" includes flow that is conveyed overland a distance of 200 feet or less from the project to the ESA, or conveyed in a pipe or open channel any distance as an isolated flow from the project to the ESA (i.e. not commingled with flows from adjacent lands). ☐ Yes ☒ No

8. **New development or redevelopment projects of a retail gasoline outlet (RGO) that create and/or replaces 5,000 square feet of impervious surface.** The development project meets the following criteria: (a) 5,000 square feet or more or (b) has a projected Average Daily Traffic (ADT) of 100 or more vehicles per day. ☐ Yes ☒ No

9. **New development or redevelopment projects of an automotive repair shops that creates and/or replaces 5,000 square feet or more of impervious surfaces.** Development projects categorized in any one of Standard Industrial Classification (SIC) codes 5013, 5014, 5541, 7532-7534, or 7536-7539. ☐ Yes ☒ No

10. **Other Pollutant Generating Project.** The project is not covered in the categories above, results in the disturbance of one or more acres of land and is expected to generate pollutants post construction, such as fertilizers and pesticides. This does not include projects creating less than 5,000 sf of impervious surface and where added landscaping does not require regular use of pesticides and fertilizers, such as slope stabilization using native plants. Calculation of the square footage of impervious surface need not include linear pathways that are for infrequent vehicle use, such as emergency maintenance access or bicycle pedestrian use, if they are built with pervious surfaces of if they sheet flow to surrounding pervious surfaces. ☐ Yes ☒ No

PART F: Select the appropriate category based on the outcomes of PART C through PART E.

1.	The project is **NOT SUBJECT TO PERMANENT STORM WATER REQUIREMENTS.**	☐
2.	The project is a **STANDARD DEVELOPMENT PROJECT.** Site design and source control BMP requirements apply. See the Storm Water Standards Manual for guidance.	☒
3.	The project is **PDP EXEMPT.** Site design and source control BMP requirements apply. See the Storm Water Standards Manual for guidance.	☐
4.	The project is a **PRIORITY DEVELOPMENT PROJECT.** Site design, source control, and structural pollutant control BMP requirements apply. See the Storm Water Standards Manual for guidance on determining if project requires a hydromodification plan management	☐

MICHAEL L. SMITH	PROJECT ENGINEER
Name of Owner or Agent *(Please Print)*	Title
[signature]	10/17/2017
Signature	Date

HYDROLOGY REPORT
FOR
8230 PRESTWICK DRIVE
SAN DIEGO, CA 92037
APN: 346-262-06
DATE: NOVEMBER 08, 2018

PREPARED BY:
SAN DIEGO LAND SURVEYING AND ENGINEERING INC.
9665 CHESAPEAKE DRIVE, SUITE 445
SAN DIEGO, CA. 92123

CITY OF SAN DIEGO
PTS 603740

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APPENDICES

EXHIBITS

PROJECT DESCRIPTION:

EXISTING PROJECT SITE DESCRIPTION:
The site is 0.4706 acres in size and is occupied by a single family residence, concrete paved driveway and landscaping. The site drains to the west, down a steep slope to a public alley. See “EXHIBIT “A”, EXISTING CONDITIONS” at the end of this report.

The impervious area of the existing site is 9,200 sf.
Percentage of site coverage is 44.9%

PROPOSED PROJECT DESCRIPTION:
Earth work will consist of grading for the basement level, removal of all existing walls and hardscape. Construct a new multi-level single family home. The disturbed area for this project is 11,900 sf. Or 0.2212 acres. Roof drains will be directed to landscaped areas on the east and west side of the project. These planters will be equipped with grated landscape inlets. The inlets will be connected to a private 6” PVC storm drain pipe which will flow down the existing slope to an existing discharge point in the public alley. This is a Standard Development Project and Hydro-modification will not be required. See “EXHIBIT “B”, PROPOSED CONDITIONS” at the end of this report.

The impervious area of the proposed site is 9,634 sf.
Percentage of site coverage is 47.0%

STANDARDS AND METHODS

PURPOSE OF CALCULATIONGS:

Compare the “pre” and “post” construction storm drain runoff quantities. Determine the adequacy of any storm drain collection system.

HYDROLOGIC MODEL AND METHODS USED:

This report uses the “Rational Method” as demonstrated in the City of San Diego Storm Drain Manual.

$$Q = CIA$$

STORM WATER DESIGN STORM:

The design storm for private site storm drain facilities shall be the 50 year storm. The design storm for public flows shall be the 100 year storm.

PRE-DEVELOPMENT RUNOFF VOLUMES AND PEAK FLOWS:

Runoff factor "C" for single-family lots with a soil type of "D" from the City of San Diego Transportation and Storm Water Design Manuals and in Appendix B is 0.55. See Exhibit "A" for plan view of the drainage area.

Time of concentration for a travel distance of 206', a drop of 72' for a slope of 35% and a C value of .55, from formula on page A-8 of said manual and included in Appendix C.

$$T = \frac{1.8(1.1-.55)\ \text{sq. root of } 206}{\text{Cubed root of } 35} = 4.4 \text{ mins.}$$

Use T = 5 min. minimum

INTENSITY-DURATION-FREQUENCY CURVES from the chart in Appendix "D". Determine rainfall intensity "I". For 5 min., 50 year storm, the rainfall intensity **I = 4.20.** For 5 min., 100 year storm, the rainfall intensity **I = 4.40.**

Zone: Existing Area ZONE E1 = 0.4706 acres
Q50 = CIA = .55 x 4.20 x 0.4706 = 1.09 CFS

Zone: Existing Area ZONE E1 = 0.4706 acres
Q100 = CIA = .55 x 4.40 x 0.4706 = 1.14 CFS

Total runoff flowing to the public alley, for the existing condition, is **1.14 CFS.**

POST-PROJECT RUNOFF VOLUMES AND PEAK FLOWS:

Runoff factor "C" for single-family lots with a soil type of "D" from the City of San Diego Transportation and Storm Water Design Manuals and in Appendix B is 0.55. See Exhibit "B" for plan view of the drainage area.

Time of concentration for a travel distance of 206’, a drop of 72’ for a slope of 35% and a C value of .55, from formula on page A-8 of said manual and included in Appendix C.

$$T = \frac{1.8(1.1-.55)\ \text{sq. root of } 206}{\text{Cubed root of } 35} = 4.4 \text{ mins.}$$

Use T = 5 min. minimum

INTENSITY-DURATION-FREQUENCY CURVES from the chart in Appendix “D”. Determine rainfall intensity “I”. For 5 min., 50 year storm, the rainfall intensity **I = 4.20.** For 5 min., 100 year storm, the rainfall intensity **I = 4.40.**

Zone: Proposed Area ZONE P1 = 0.4706 acres
Q50 = CIA = .55 x 4.20 x 0.4706 = 1.09 CFS
Q100 = CIA = .55 x 4.40 x 0.4706 = 1.14 CFS

Total runoff flowing to the public alley, for the proposed condition, is **1.14 CFS.**

Existing private 6” PVC pipe drains the existing developed portion of the site and discharges to an existing improve public alley.

Pipe Capacity:
6” PVC pipe”
N = .013 S = 61.41% Q req. = 1.09 CFS
Dn= 5.88” V = 5.63 f/s

The proposed developed area is basically the same size and will also be drained by the existing pipe.

CONCLUSION:

There is no increase in runoff over the existing condition. No damage to the adjacent or downstream private property or public improvements is anticipated. The storm water from the improved public alley flows to and down Calle De Oro to an existing grated inlet at the intersection of Calle De Oro and El Paseo Grande. It then discharges to the Pacific Ocean.

This project is not required to obtain approval from the Regional Water Quality Control Board under Federal Clean Water Act (CWA) section 401 or 404 as it does not discharge dredged or fill material into waters of the United States, including wetlands.

CURRENT CITY REGULATIONS:
Pursuant to San Diego Municipal Code Chapter 14 Article 2 Division 2, Storm Water Runoff and Drainage Regulations, drainage regulations apply to all development in the City of San Diego, whether a permit or other approval is required. Drainage design policies and procedures for the City of San Diego are given in the City of San Diego's "Drainage Design Manual" which is incorporated in the Land Development Manual as Appendix B.

Storm Water Quality
Pursuant to Section 402 of the Clean Water Act (CWA), the EPA has established regulations under the National Pollutant Discharge Elimination System (NPDES) program to control direct storm water discharges. In California, the State Water Resources Control Board (SWRCB) administers the NPDES permitting programs and is responsible for developing waste discharge requirements. The California Regional Water Quality Control Board San Diego Regional (SDRWQCB) also is responsible for developing waste discharge requirements specific to its jurisdiction.

Municipal Strom Water Permit:
The current municipal storm water permit (2013 MS4 Permit) for Region 9 Order No. R9-2013-0001, was adopted on May 8, 2013 by the San Diego Regional Water Quality Control Board (Regional Board) and became effective on June 27, 2013. This order was amended by adoption of Order No. R9-2015-0001 on February 11, 2015 and adoption of Order No R9-2015-0100 on November 18, 2015. This is an update to the 2007 MS4 Permit, Order No. R9-2007-0001. The implementation of the 2013 MS4 Permit criteria and updates to the City of San Diego Storm Water Standards (based on the Copermittee's Model BMP Design Manual) took place on February 16, 2016.

Projects less than one acre in size, and not part of a larger common plan of development, are not subject to the requirements of the General Construction Permit. However, in the City of San Diego, construction storm water requirements apply to all new development activities based on the City of San Diego's Storm Water Management and Discharge Control Ordinance (San Diego)Municipal Code Section 43.03, et. Seq.) Projects less than one acre are required to have a Water Pollution Control Plan (WPCP) which identifies the pollution prevention measures that will be taken.

Temporary Groundwater Extraction:
The San Diego Water Board has adopted a NPDES Permit that cover groundwater extraction discharges to surface waters in the San Diego Region. Discharges to bodies within the San Diego Region including surface waters, estuaries, and the Pacific Ocean (Order No. R9-2008-0002, NPDES No. CAG919002.

This project in covered under the above regulations.

CERTIFICATION STATEMENT:

This Hydrology Report has been prepared under the direction of the following registered civil engineer. The registered civil engineer (Engineer) attests to the technical information contained herein and the engineering data upon which the following design, recommendations, conclusions and decisions are based. The selection, sizing, and design of storm water treatment and other control measures in this report meet the requirements of the Regional Water Quality Control Board Order R9-2007-0001 and subsequent amendments.

ENGINEER OF WORK:

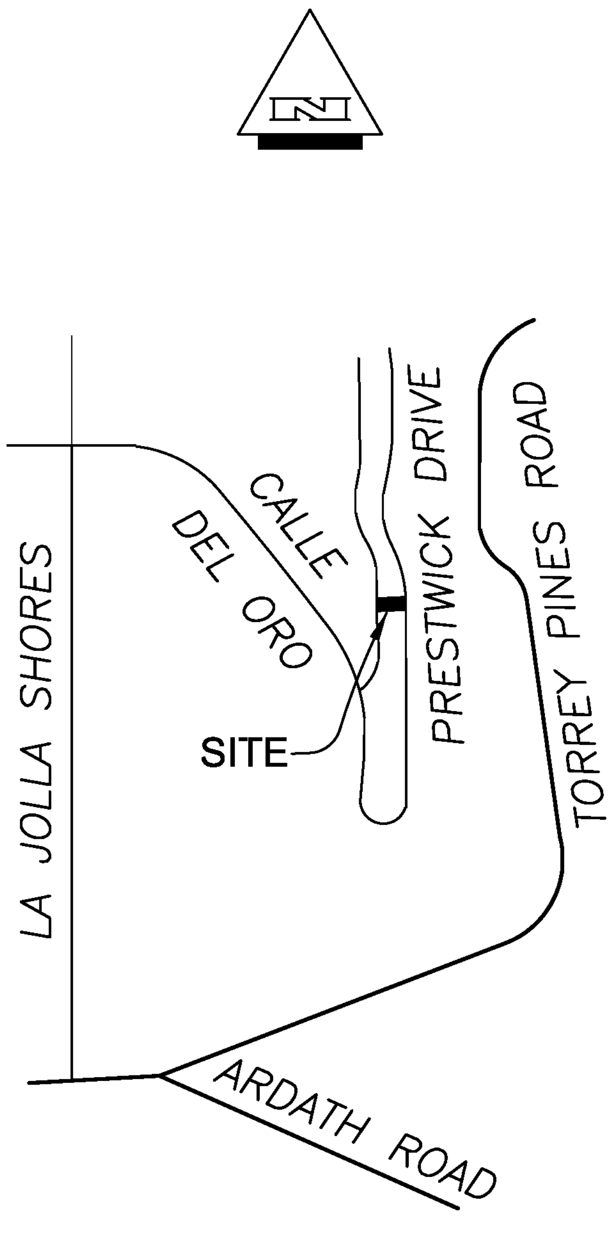
____________________________ DATE: 11-08-2018

MICHAEL LEE SMITH, RCE 35471
MY REGISTRATION EXPIRES ON
9/30/2019

REGISTERED PROFESSIONAL ENGINEER
MICHAEL L. SMITH
R.C.E. 35471
STATE OF CALIFORNIA

APPENDIX A

VICINITY MAP



VICINITY MAP

NO SCALE

APPENDIX B

CITY OF SAN DIEGO STORM DRAIN MANUAL
TABLE A-1, RUNOFF COEFFICIENTS

Table A-1. Runoff Coefficients for Rational Method

Land Use	Runoff Coefficient (C) Soil Type [1]
Residential:	
Single Family	0.55
Multi-Units	0.70
Mobile Homes	0.65
Rural (lots greater than ½ acre)	0.45
Commercial [2]	
80% Impervious	0.85
Industrial [2]	
90% Impervious	0.95

Note:
[1] Type D soil to be used for all areas.
[2] Where actual conditions deviate significantly from the tabulated imperviousness values of 80% or 90%, the values given for coefficient C, may be revised by multiplying 80% or 90% by the ratio of actual imperviousness to the tabulated imperviousness. However, in case shall the final coefficient be less than 0.50. For example: Consider commercial property on D soil.

Actual imperviousness		=	50%
Tabulated imperviousness		=	80%
Revised C	= (50/80) x 0.85	=	0.53

The values in Table A-1 are typical for urban areas. However, if the basin contains rural or agricultural land use, parks, golf courses, or other types of nonurban land use that are expected to be permanent, the appropriate value should be selected based upon the soil and cover and approved by the City.

A.1.3. Rainfall Intensity

The rainfall intensity (I) is the rainfall in inches per hour (in/hr.) for a duration equal to the T_C for a selected storm frequency. Once a particular storm frequency has been selected for design and a T_C calculated for the drainage area, the rainfall intensity can be determined from the Intensity-Duration-Frequency Design Chart (Figure A-1).

APPENDIX C

CITY OF SAN DIEGO STORM DRAIN MANUAL
TABLE A-4, OVERLAND TIME OF FLOW

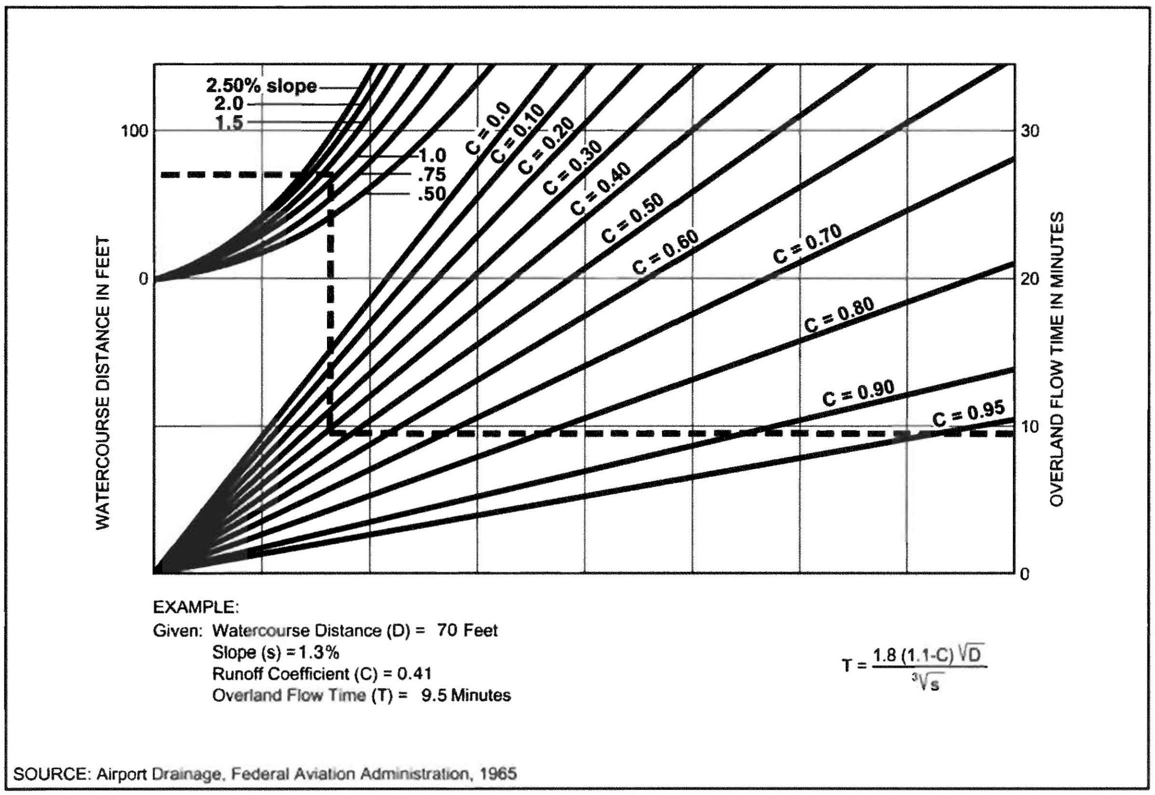


Figure A-4. Rational Formula – Overland Time of Flow Nomograph

Note: Use formula for watercourse distances in excess of 100 feet.

APPENDIX D

CITY OF SAN DIEGO STORM DRAIN MANUAL
FIGURE A-1, INTENSITY DURATION DESIGN CHART

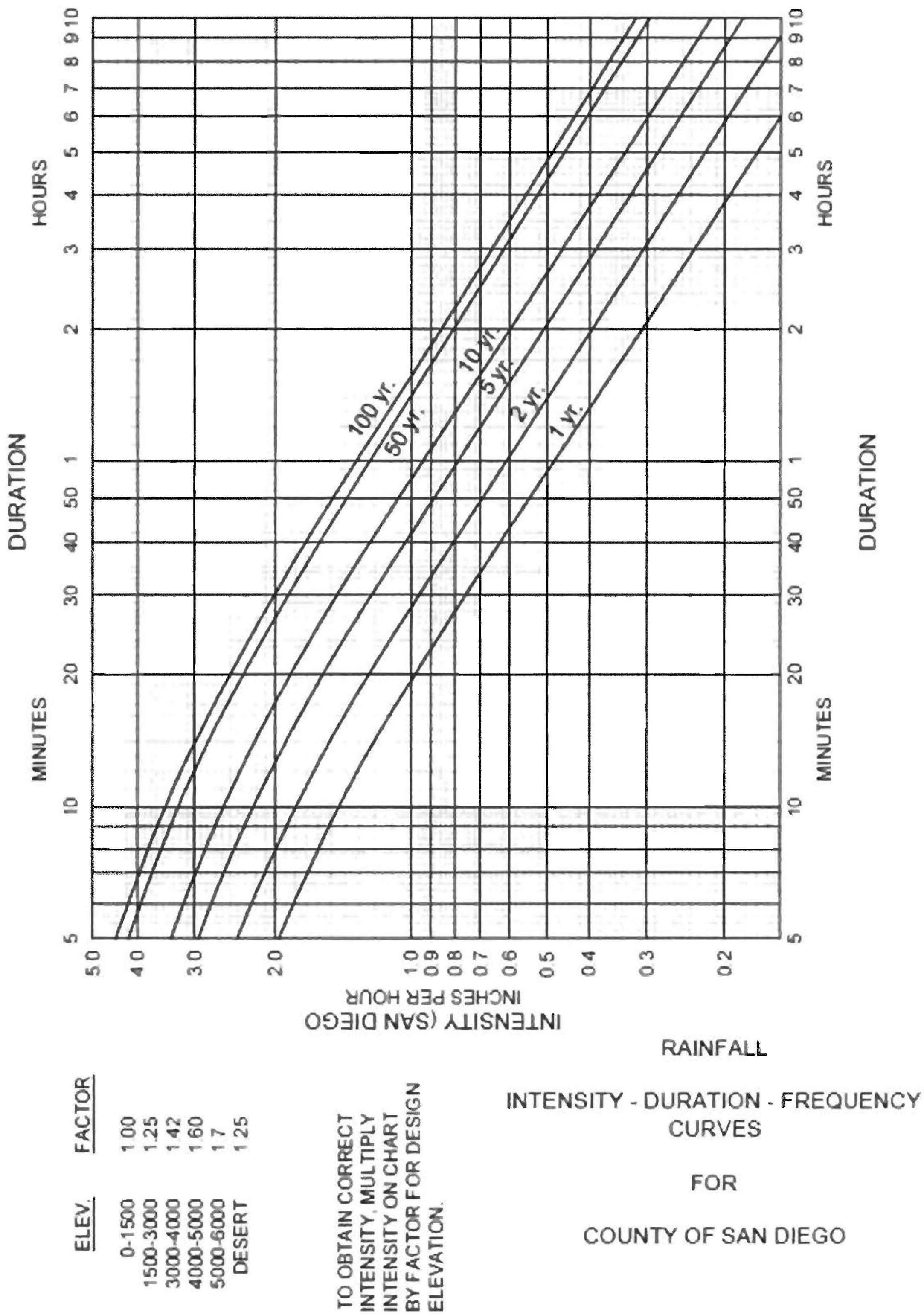


Figure A-1. Intensity-Duration-Frequency Design Chart

APPENDIX E

PRIVATE EXISTING 6″ PVC PIPE CALCULATIONS

Culvert Report

EXISITNG PRIVATE 6 INCH PVC PIPE AT 61.31%

Invert Elev Dn (ft)	= 252.91
Pipe Length (ft)	= 86.31
Slope (%)	= 61.41
Invert Elev Up (ft)	= 305.91
Rise (in)	= 6.0
Shape	= Circular
Span (in)	= 6.0
No. Barrels	= 1
n-Value	= 0.013
Culvert Type	= Circular Culvert
Culvert Entrance	= Smooth tapered inlet throat
Coeff. K,M,c,Y,k	= 0.534, 0.555, 0.0196, 0.9, 0.2

Embankment

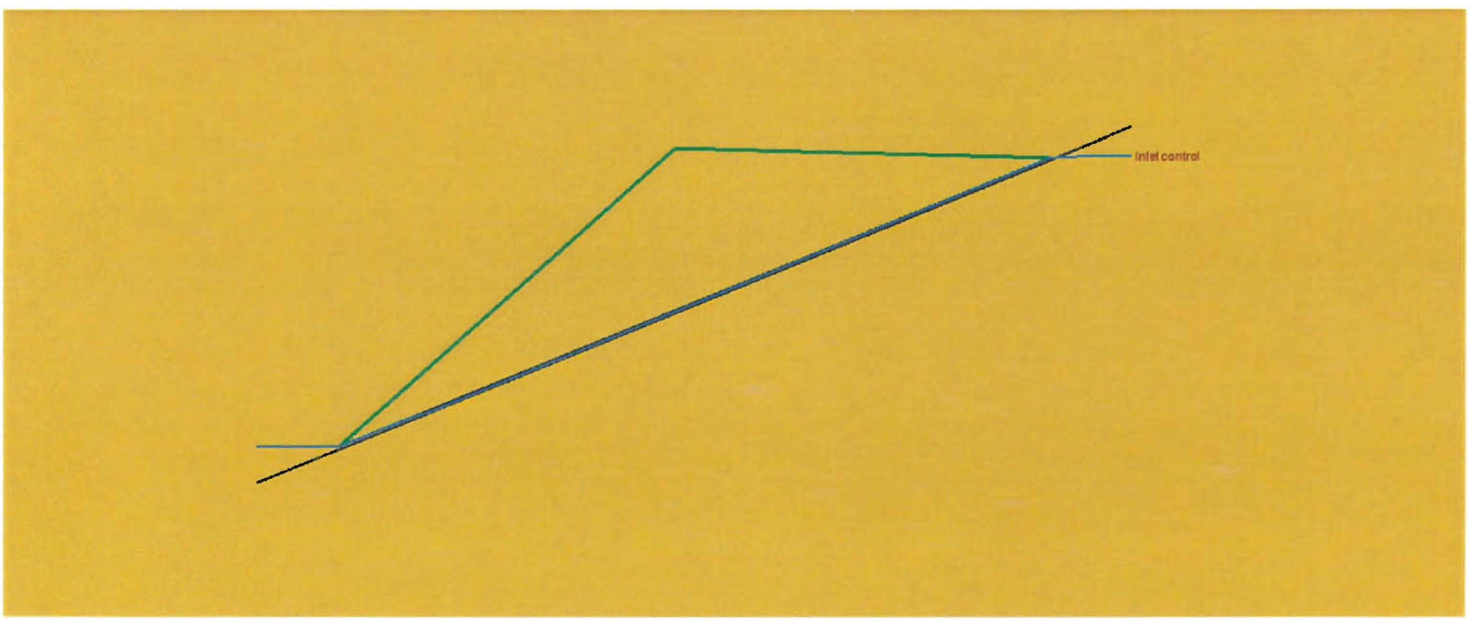
Top Elevation (ft)	= 308.00
Top Width (ft)	= 5.00
Crest Width (ft)	= 5.00

Calculations

Qmin (cfs)	= 0.50
Qmax (cfs)	= 2.00
Tailwater Elev (ft)	= (dc+D)/2

Highlighted

Qtotal (cfs)	= 1.10
Qpipe (cfs)	= 1.10
Qovertop (cfs)	= 0.00
Veloc Dn (ft/s)	= 5.63
Veloc Up (ft/s)	= 5.68
HGL Dn (ft)	= 253.40
HGL Up (ft)	= 306.39
Hw Elev (ft)	= 306.82
Hw/D (ft)	= 1.82
Flow Regime	= Inlet Control



Q			Veloc		Depth	
Total	Pipe	Over	Dn	Up	Dn	Up
(cfs)	(cfs)	(cfs)	(ft/s)	(ft/s)	(in)	(in)
0.50	0.50	0.00	2.78	3.30	5.16	4.32
0.60	0.60	0.00	3.24	3.62	5.36	4.72
0.70	0.70	0.00	3.70	3.96	5.53	5.06
0.80	0.80	0.00	4.17	4.34	5.66	5.33
0.90	0.90	0.00	4.64	4.76	5.76	5.53
1.00	1.00	0.00	5.13	5.21	5.83	5.67
1.10	1.10	0.00	5.63	5.68	5.88	5.76
1.20	1.20	0.00	6.13	6.16	5.92	5.83
1.30	1.30	0.00	6.63	6.65	5.94	5.88
1.40	1.40	0.00	7.14	7.15	5.95	5.91
1.50	1.50	0.00	7.65	7.66	5.96	5.93
1.60	1.60	0.00	8.15	8.16	5.97	5.95
1.70	1.70	0.00	8.66	8.67	5.98	5.96
1.80	1.80	0.00	9.17	9.17	5.98	5.97
1.90	1.88	0.02	9.59	9.60	5.99	5.97

HGL			
Dn	Up	Hw	Hw/D
(ft)	(ft)	(ft)	
253.34	306.27	306.45	1.09
253.36	306.30	306.51	1.20
253.37	306.33	306.57	1.31
253.38	306.35	306.62	1.41
253.39	306.37	306.66	1.51
253.40	306.38	306.71	1.61
253.40	306.39	306.82	1.82
253.40	306.40	306.94	2.06
253.40	306.40	307.07	2.31
253.41	306.40	307.20	2.59
253.41	306.40	307.35	2.88
253.41	306.41	307.51	3.20
253.41	306.41	307.68	3.53
253.41	306.41	307.85	3.89
253.41	306.41	308.01	4.20

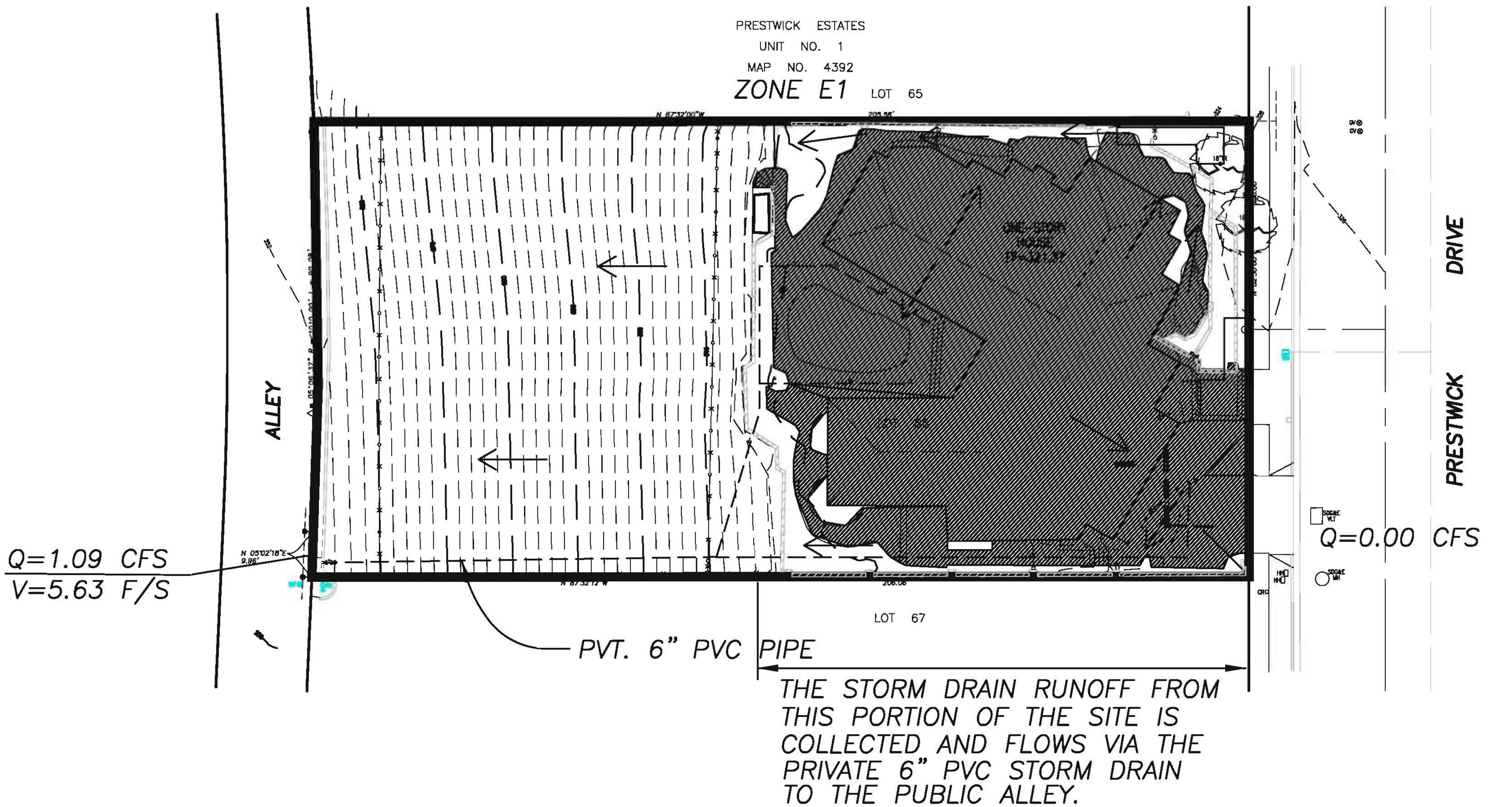
EXHIBIT “A”

EXISTING CONDITIONS

EXHIBIT A

HYDROLOGY MAP
EXISTING CONDITIONS

SCALE 1" = 30'



PROJECT AREA
AREA = 20,500 SF. OR 0.4706 AC.

ZONE E1
AREA = 20,500 SF. OR 0.4706 AC.

IMPERVIOUS AREA
AREA = 9,200 SF. OR 0.2112 AC.
44.9% OF SITE

EXHIBIT “B”

PROPOSED CONDITIONS

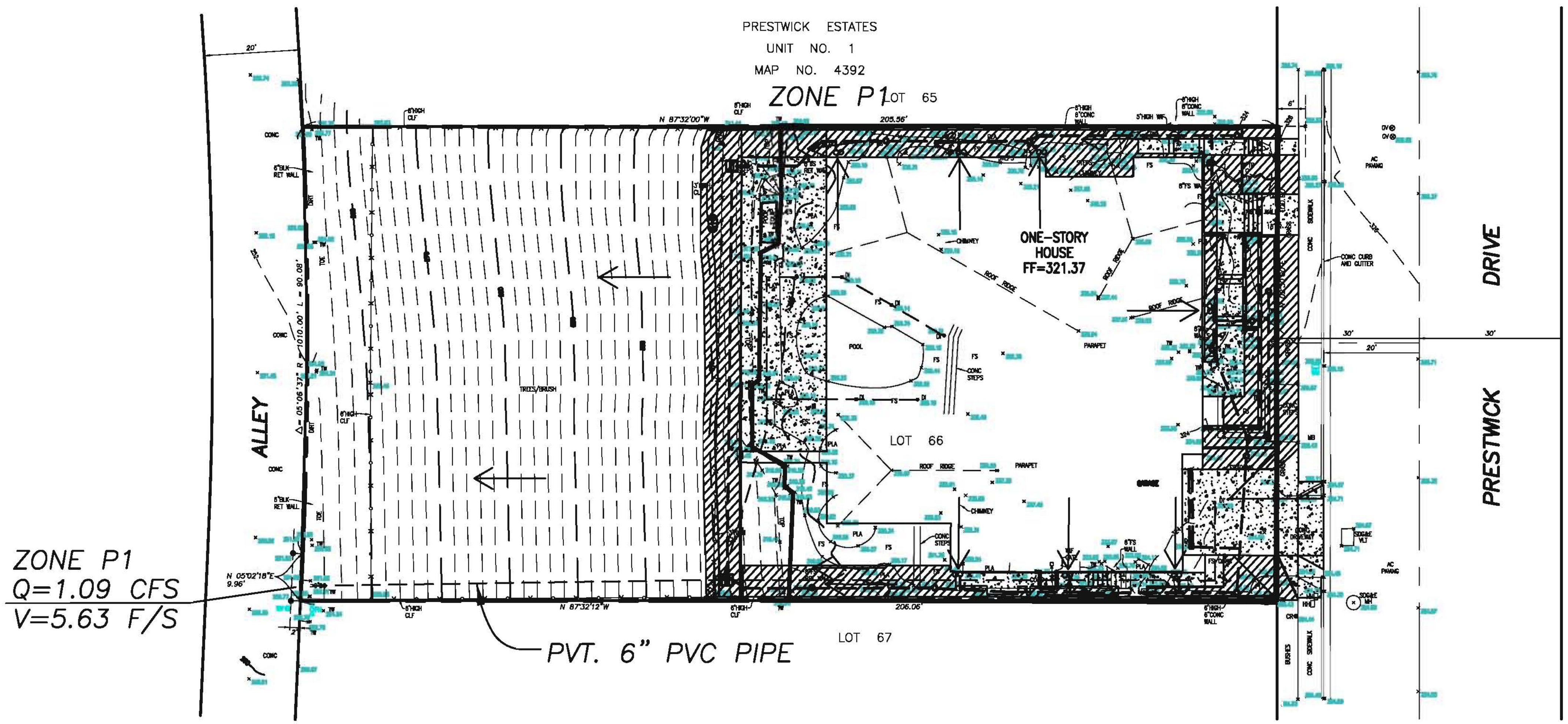


EXHIBIT B

PROPOSED IMPERVIOUS

SCALE 1" = 30'

PROJECT AREA
AREA = 20,500 SF. OR 0.4706 AC.

ZONE P1
AREA = 20,500 SF. OR 0.4706 AC.

IMPERVIOUS AREA
AREA = 9,634 SF. OR 0.2212 AC.
47.0% OF SITE