HYDROLOGY
and
HYDRAULICS REPORT

for
The Klein Residence

Prepared for:
City of San Diego
Development Services Department
1222 First Ave. MS 501
San Diego, CA 92101-4155

Project Site Address
2585 Calle Del Oro
La Jolla, CA 92037

Study Prepared by:
Omega Engineering Consultants
4340 Viewridge Ave Suite B
San Diego, CA 92123
(858) 634-8620

Preparation Date
May 8th, 2017

Patric de Boer

Patric de Boer **RCE 83583**
Registration Expires **3-31-2019**

REGISTERED PROFESSIONAL ENGINEER
PATRIC T. DE BOER
No. C 83583
EXP. 3/31/19
CIVIL
STATE OF CALIFORNIA

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SITE AND PROJECT DESCRIPTION

This Hydrology and Hydraulics report has been prepared as part of the Grading and Development Plan set for the project located at 2585 Calle Del Oro. The project site is currently occupied by a 3,500 sq. ft. single family residence and proposes demolishing the existing structure and building a new single family residence. The proposed design implements a drainage system that conveys runoff to bioretention area and then a pump vault. The pump system will convey stormwater via force main system to the curb along Calle del Oro

The site drainage basin is located in the Scripps Hydrologic Area of the Peñasquitos Hydrologic Unit of the San Diego Hydraulic Region (906.30).See Figure No. 1 for the vicinity map. See Figure No. 2 for the existing drainage limits. See Figure No. 3 for the proposed drainage limits.

METHODOLOGY

This drainage report has been prepared in accordance with current City of San Diego regulations and procedures. All of the proposed pipes and facilities have been designed to intercept and convey the 100-year storm. The Modified Rational Method was used to compute the anticipated runoff. See the attached calculations for particulars. The following references have been used in preparation of this report:

(1) City of San Diego Hydrology Manual, April, 1984.
(2) Handbook of Hydraulics, E.F. Brater & H.W. King, 6th Ed., 1976.
(3) Modern Sewer Design, American Iron & Steel Institute, 1st Ed., 1980.
(4) County of San Diego Hydrology Manual, June, 2003

RATIONAL METHOD

Q=CIA

Where:

Q = peak discharge, in cubic feet per second (cfs)
C = runoff coefficient, proportion of the rainfall that runs off the surface (no units)
= (0.90*(% impervious) + C_p*(1-% Impervious))
I = average rainfall intensity for a duration equal to the Tc for the area, (in/hr)
= $7.44*P_6*T_c^{-0.645}$
A = drainage area contributing to the design location, in acres
C_p = Pervious Coefficient Runoff Value, from County Hydrology Manual Appendix A
= 0.35 for Type D soil
T_c = $\frac{1.8\,(1.1-C)*(T_c)^{0.5}}{S^{0.33}}$
S = Slope of drainage course

EXISTING SITE CONDITIONS:

The existing site consists of a single family residence located on a site that is approximately 0.45 acres and 46% impervious, with landscaped areas around the periphery. Existing drainage is facilitated via overland flow. The property is divided into two drainage basins EX-1 & EX-2. The westerly basin EX-1 flows west via surface flow to the neighbor’s lot and eventually being intercepted by a curb inlet along Valecitos Rd. Flow from the easterly basin flows northeast to the gutter of Calle Del Oro. From there, it flows in along the street in a westerly direction until it is intercepted by a gutter at the corner of Calle del Cielo and Calle del Oro.

DEVELOPED SITE CONDITIONS:

The project proposes the removal of the existing structure and hardscape and the construction of a new multistory single family home in its place. The proposed building foot print will be 5,100 sf. The proposed site will be 51% impervious. Onsite drainage patterns will be modified due to surface modifications. A new storm drain system shall convey the majority of onsite runoff a pump vault in the back yard. The pump system will convey the runoff via force main to the gutter along Calle del Oro. Runoff that is pumped to the curb will follow the existing offsite flow path the curb inlet at the several hundred yards north of the site on Calle Del Oro. Drainage from a small portion of the site below the retaining wall be allowed to continue draining to the neighboring lot.

EXISTING RUNOFF ANALYSIS:

The Rational Method was used for calculating existing peak flow rates for the 85th %, and 100-year storms. Analysis of the existing conditions breaks the disturbed area into two separate drainage areas each with a separate discharge point. Runoff coefficients in the range of 0.43-0.85 were used for the existing basins.

DEVELOPED RUNOFF ANALYSIS:

The Rational Method was used for calculating proposed peak flow rates for the 85th% and 100-year storms. Analysis of the proposed site breaks the disturbed area into two separate drainage areas. The westerly basin (A-1) will surface flow to the existing discharge point along the westerly boundary of the site. The easterly basin (B-1) will discharge to the curb via a force main systems. Runoff coefficients in the range of 0.35-0.69 were used for the proposed basins.

RESULTS AND CONCLUSIONS

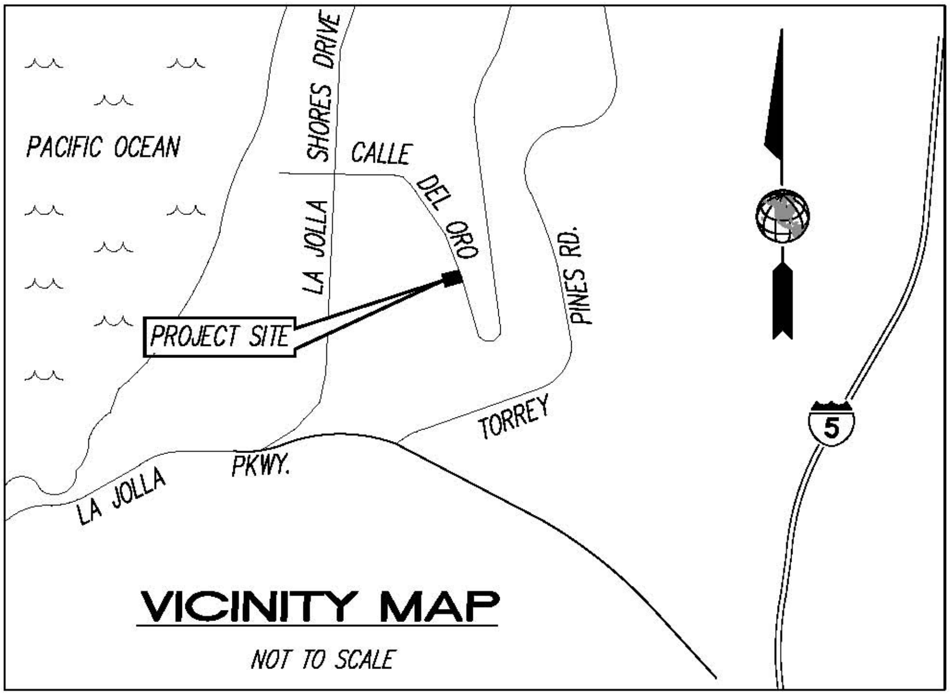
The redevelopment of the site will alter onsite drainage patterns by nearly eliminating overland flow to the downhill neighboring lots. Runoff will instead by discharge to the curb Calle del Oro by a forcemain pump system. The project will increase the amount of stormwater generated by the site, but the actual discharge rates from the site will be reduced. This is because the force main pumps will be limited to a constant outflow rate that is less than the existing discharge at Discharge Pt. #2. The pump vaults have been sized to store stormwater in excess of the discharge capacities of the pumps during the peak of high intensity storm events

The project will decrease the flow to the downhill areas west of the site and decrease the discharge to the curb from 0.81 to 0.38 cfs.

No CWA 401 or 404 permit is required as the project will not discharge fill or dredge material to a water of the state or US. The site will not dredge or place any fill in a water of the state or US.

Site design complies with ASBS requirements by draining all low flow runoff to biofiltration area. See separate SWQMP.

It is the opinion of Omega Engineering Consultants that this project will not negatively effect the downstream waterways and receiving water bodies.


PACIFIC OCEAN
LA JOLLA SHORES DRIVE
CALLE DEL ORO
PINES RD.
PROJECT SITE
TORREY
LA JOLLA PKWY.
5
VICINITY MAP
NOT TO SCALE

Klein Residence

12/7/2016

HYDROLOGY AND HYDRAULICS CALCS (Table No. 1)

BASIN	AREA (SF)	AREA (AC)	% Imp	"C" Value
EX-1	11,539	0.26	15.0%	0.43
EX-2	7,950	0.18	90.0%	0.85
EX. TOTAL	**19,489**	**0.45**		
A-1	3,317	0.08	0.0%	0.35
B-1	16,809	0.39	61.1%	0.69
PROP TOTAL	**20,126**	**0.46**		

Basin Confluence	Symbol
-	-
-	-

(A) "CP#1" Confluence Point Number 1

(B) C value for bare ground is 0.35 (Table 3-1 County Hydrology Manual)
C value for impervious surfaces is 0.9
Basins with mixed surface type use a weighted average
of these 2 values. (impervious % x 0.9)+(pervious % x 0.35)

Ex. and proposed areas totals are different as the exisiting and proposed sites have pools of different sizes

Klein Residence

12/7/2016

HYDROLOGY AND HYDRAULICS CALCS (Table No. 2)

Sub-Basin	AREA Ac.	"C"	CA	L (ft) Travel	H (ft) (elev)	S(%) (avg.)	Tc min.	T tot mins	I in/hr	Q cfs	Q tot cfs	L (ft) (Pipe)	S (%) (Pipe)	Dia. (in)	K'	D\d	pipe #	NOTES 85th % storm
EX-1	0.26	0.43	0.11	150	40.00	26.67	5.0	5.00	0.20	0.02	0.02							
							Discharge Pt #1 ex. discharge=				**0.02**	**CFS**						
EX-2	0.18	0.85	0.15	150	6.00	4.00	5.0	5.00	0.20	0.03	0.03							
							Discharge Pt #2 ex. discharge=				**0.03**	**CFS**						
A-1	0.08	0.35	0.03	15	6.00	40.00	5.0	5.00	0.20	0.01	0.01							
							Discharge Pt #1 prop. discharge=				**0.01**	**CFS**						
B-1	0.39	0.69	0.26	70	5.00	6.00	5.0	5.00	0.20	0.05	0.05							
									Pump Discharge=		0.43							
							Discharge Pt #1 prop. discharge=				**0.43**	**CFS**						

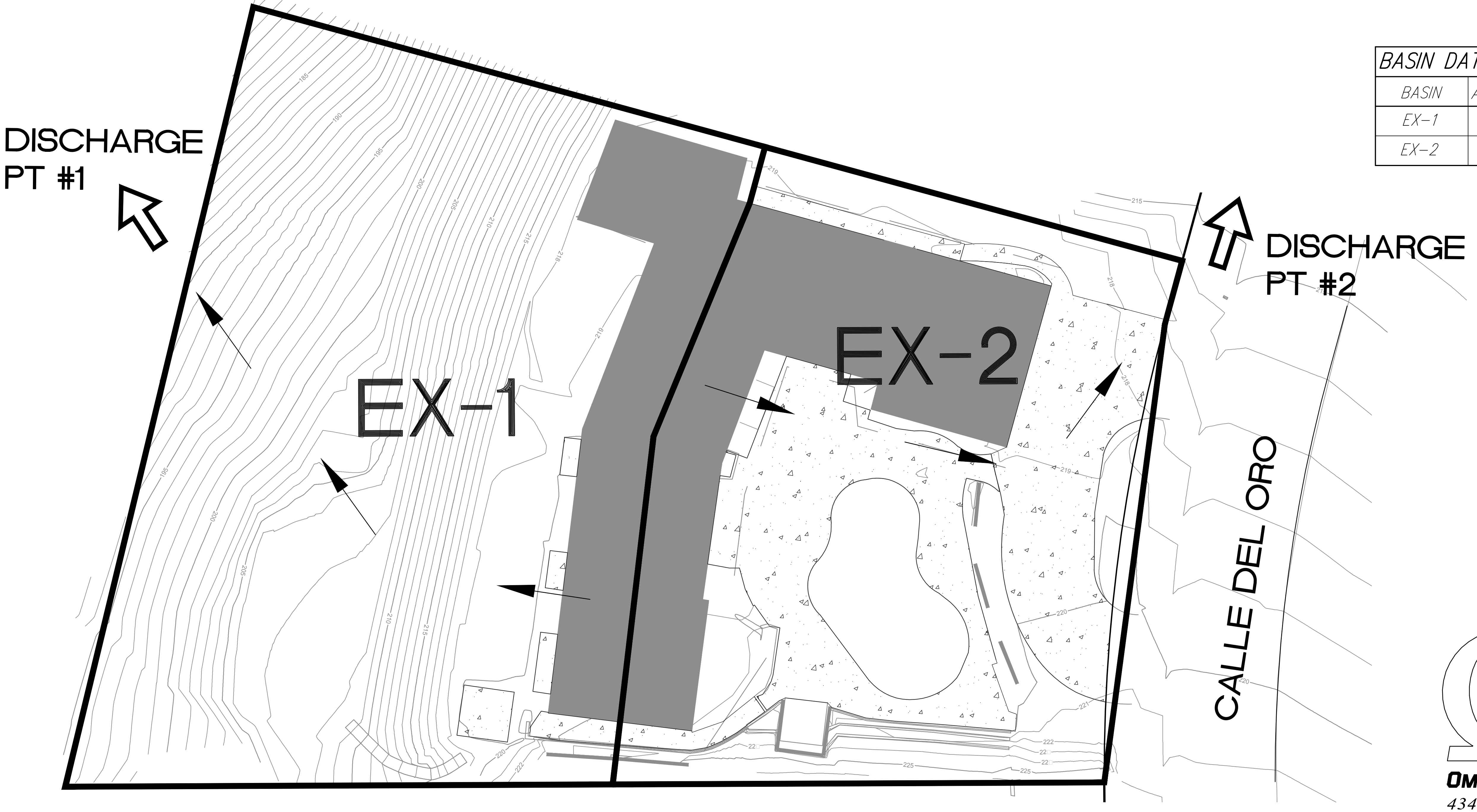
Klein Residence

HYDROLOGY AND HYDRAULICS CALCS (Table No. 2)

12/7/2016

Sub-Basin	AREA Ac.	"C"	CA	L (ft) Travel	H (ft) (elev)	S(%) (avg.)	Tc min.	T tot mins	I in/hr	Q cfs	Q tot cfs	L (ft) (Pipe)	S (%) (Pipe)	Dia. (in)	K'	D\d	pipe #	NOTES 100-yr
																	P(6)= 2.00	
EX-1	0.26	0.43	0.11	150	40.00	26.67	5.0	5.00	5.27	0.60	0.60							
							Discharge Pt #1 ex. discharge=				**0.60**	**CFS**						
EX-2	0.18	0.85	0.15	150	6.00	4.00	5.0	5.00	5.27	0.81	0.81							
							Discharge Pt #2 ex. discharge=				**0.81**	**CFS**						
A-1	0.08	0.35	0.03	15	6.00	40.00	5.0	5.00	5.27	0.14	0.14							
							Discharge Pt #1 prop. discharge=				**0.14**	**CFS**						
B-1	0.39	0.69	0.26	70	5.00	6.00	5.0	5.00	5.27	1.40	1.40							
									Pump Discharge=		0.38							
							Discharge Pt #1 prop. discharge=				**0.38**	**CFS**						

EXISTING HYDROLOGY EXHIBIT
THE KLEIN RESIDENCE

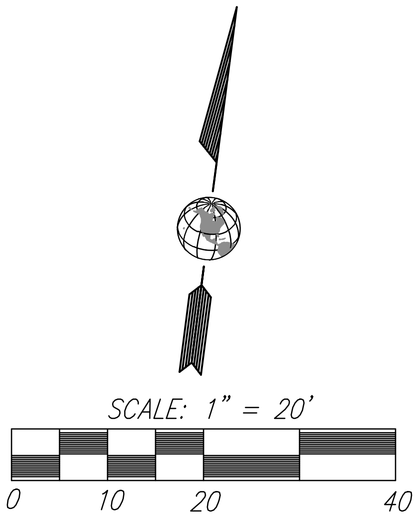


LEGEND:

AREA LIMITS	━
DRAINAGE DIRECTION ARROW	→ → →
BASIN NUMBER	A-#.#
BUILDING AREA	■
PAVEMENT AREA	▨
PERVIOUS AREA	□

BASIN DATA

BASIN	*AREA (ac)*	*IMPERVIOUS %*	*Q_{100}(cfs)*
EX-1	*0.26*	*15.3%*	*0.60*
EX-2	*0.18*	*89.7%*	*0.81*



ΩMEGA

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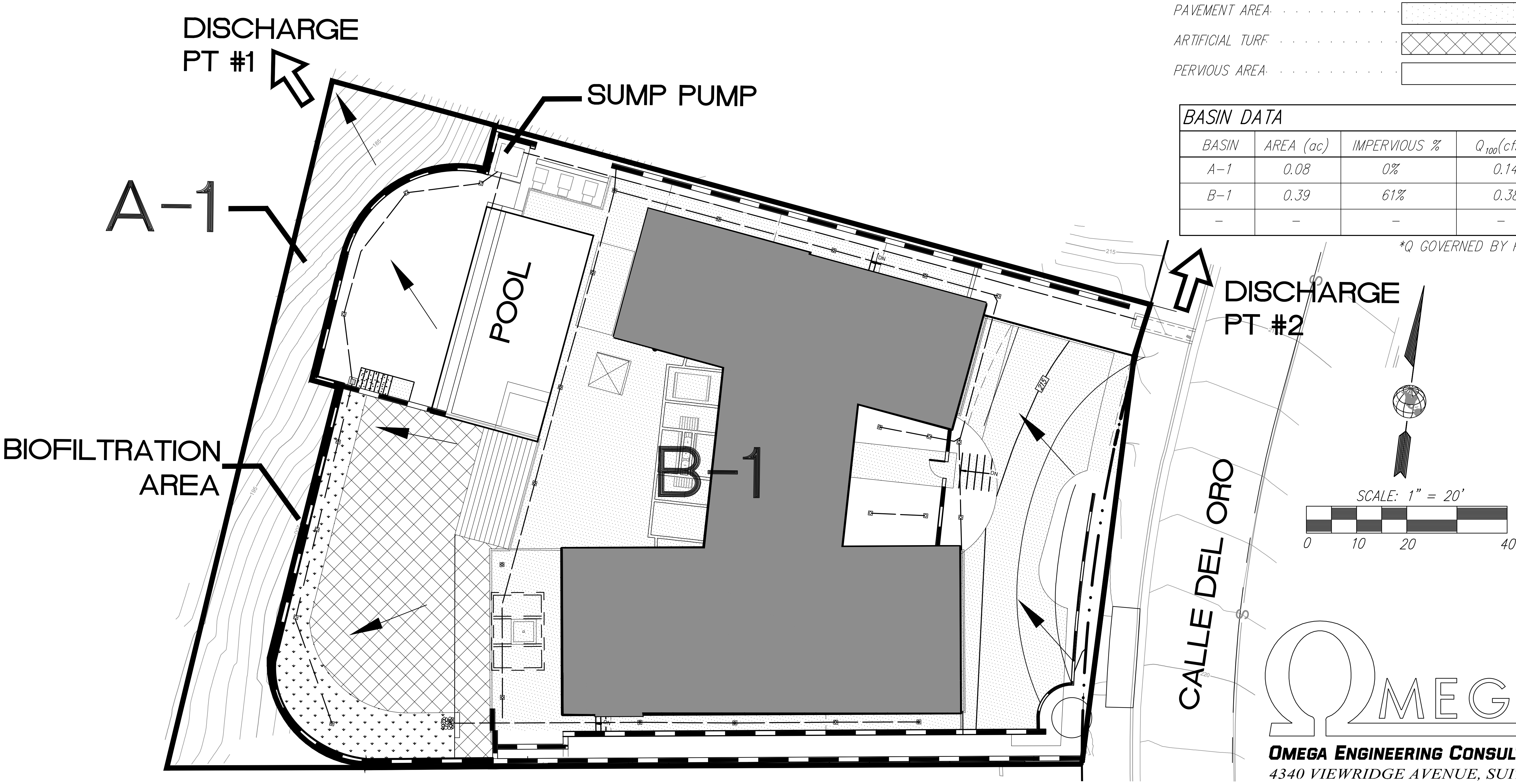
PROPOSED HYDROLOGY EXHIBIT
THE KLEIN RESIDENCE



LEGEND:

AREA LIMITS	━━━
DRAINAGE DIRECTION ARROW	→ → →
BASIN NUMBER	A-#
BUILDING AREA	■
PAVEMENT AREA	░
ARTIFICIAL TURF	▨
PERVIOUS AREA	□

BASIN DATA

BASIN	AREA (ac)	IMPERVIOUS %	Q_{100}(cfs)
A-1	0.08	0%	0.14
B-1	0.39	61%	0.38*
-	-	-	-

*Q GOVERNED BY PUMP

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Basin B-1, force main head calculation

Total length = Pipe length + fitting equivalent lengths

Pipe Length Lengths

L= 135

Equivalent Lengths

2- 90^{o} elbows @ 5.0 ft ea=	10
1- Ball Valve @ 1.2 ft ea=	1.2
Total=	11.2

Total Length

146.2

Flowrate (Q)

0.38 CFS
170.544 GPM
170.544 GPM per pipe

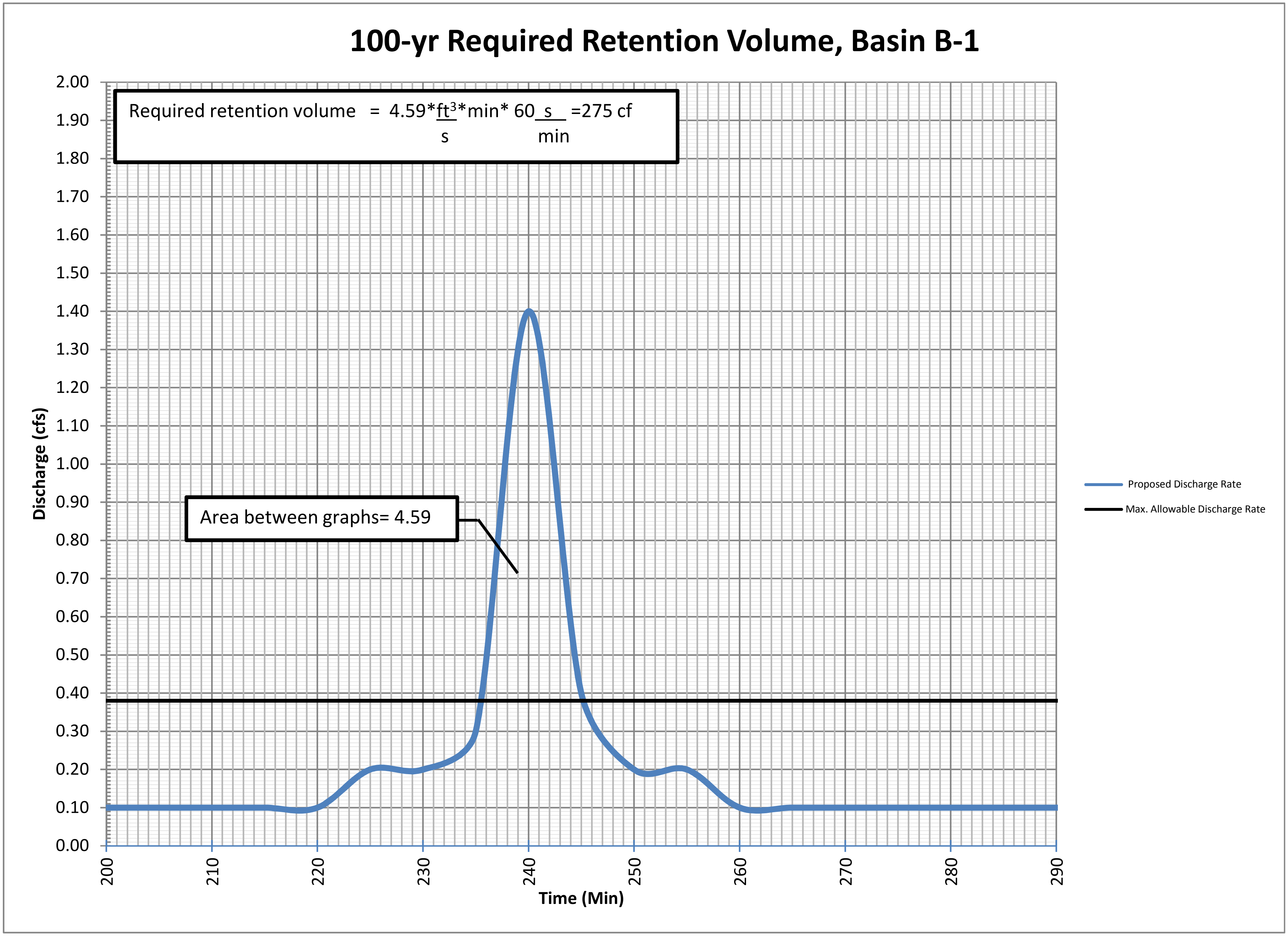
Total Head = H_{static} + $H_{friction}$

H_{static} = 28 ft

$$H_{friction} = 0.2083 \times \left(\frac{100}{C}\right)^{1.852} \times \left(\frac{Q^{1.852}}{D_h{}^{4.8655}}\right) \times \frac{L}{100} = \quad 9.32$$

C=150 Q=90 gpm D_h=3.0

Total Head = 37.32

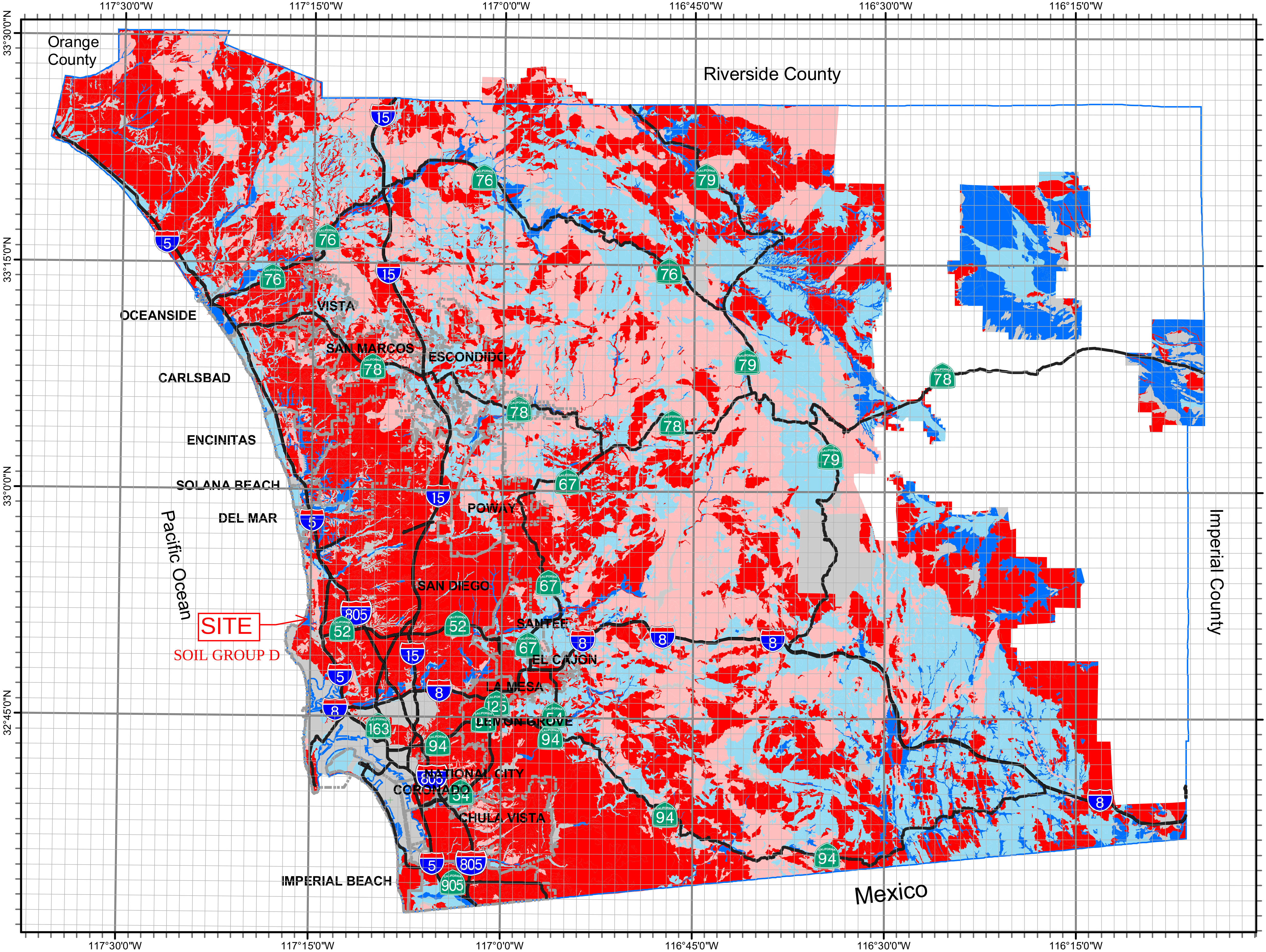


Required retention volume = $4.59*\frac{ft^3}{s}*min* 60\frac{s}{min}$ =275 cf

Area between graphs= 4.59

APPENDICES:

County of San Diego Hydrology Manual Soil Hydrologic Group



Legend

- ▬ Major Roads
- ▨ Incorporated City Bdy

HYDROLOGIC SOIL GROUP

- ⬜ Hydrologic Group Undefined
- 🟦 Hydrologic Group A
- 🟦 Hydrologic Group B
- 🟥 Hydrologic Group C
- 🟥 Hydrologic Group D
- ⬜ No Soil Data

Note: Soil Data Source
USDA/NRCS
SSURGO Soils 2007

N

3 1.5 0 3 Miles

DPW GIS

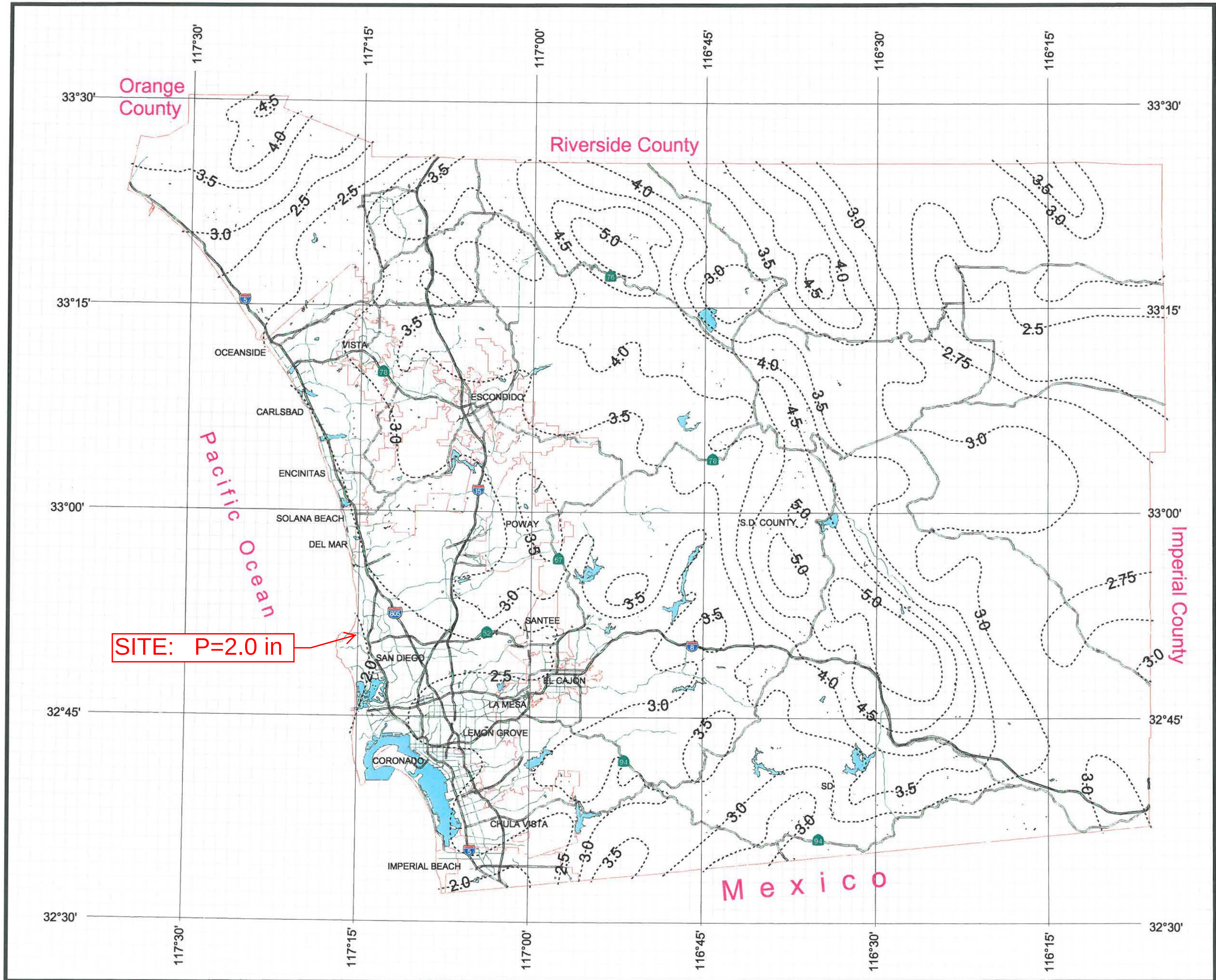
SanGIS

County of San Diego Hydrology Manual

Rainfall Isopluvials

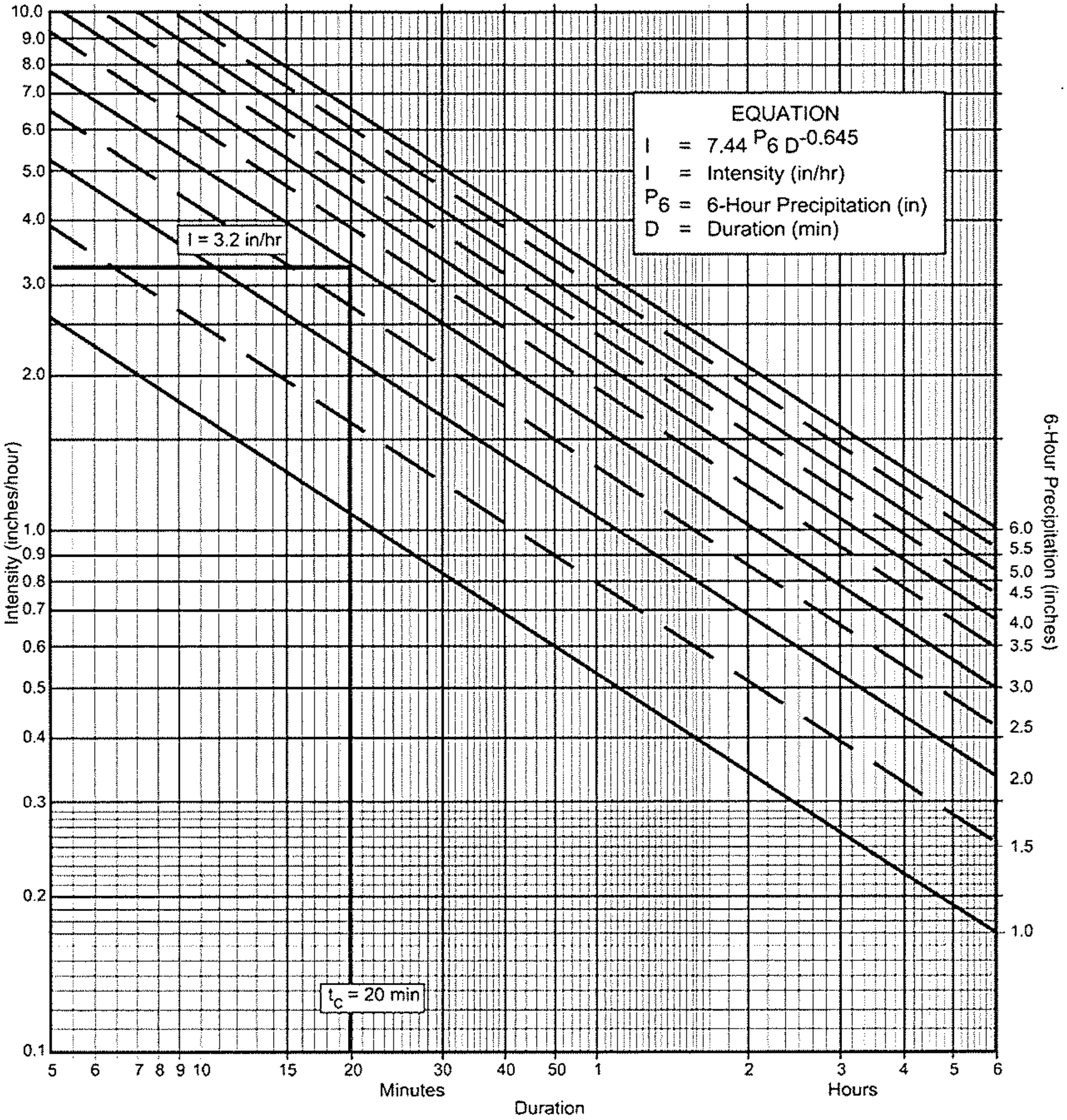
100 Year Rainfall Event - 6 Hours

....	Isopluvial (inches)



SITE: P=2.0 in

3 0 3 Miles



Directions for Application:

(1) From precipitation maps determine 6 hr and 24 hr amounts for the selected frequency. These maps are included in the County Hydrology Manual (10, 50, and 100 yr maps included in the Design and Procedure Manual).

(2) Adjust 6 hr precipitation (if necessary) so that it is within the range of 45% to 65% of the 24 hr precipitation (not applicaple to Desert).

(3) Plot 6 hr precipitation on the right side of the chart.

(4) Draw a line through the point parallel to the plotted lines.

(5) This line is the intensity-duration curve for the location being analyzed.

Application For

(a) Selected frequency ______ year

(b) P_6 = ______ in., P_{24} = ______, $\frac{P_6}{P_{24}}$ = ______ %$^{(2)}$

(c) Adjusted $P_6^{(2)}$ = ______ in.

(d) t_x = ______ min.

(e) I = ______ in./hr.

Note: This chart replaces the Intensity-Duration-Frequency curves used since 1965.

P6 Duration	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
5	2.63	3.95	5.27	6.59	7.90	9.22	10.54	11.86	13.17	14.49	15.81
7	2.12	3.18	4.24	5.30	6.36	7.42	8.48	9.54	10.60	11.66	12.72
10	1.68	2.53	3.37	4.21	5.05	5.90	6.74	7.58	8.42	9.27	10.11
15	1.30	1.95	2.59	3.24	3.89	4.54	5.19	5.84	6.49	7.13	7.78
20	1.08	1.62	2.15	2.69	3.23	3.77	4.31	4.85	5.39	5.93	6.46
25	0.93	1.40	1.87	2.33	2.80	3.27	3.73	4.20	4.67	5.13	5.60
30	0.83	1.24	1.66	2.07	2.49	2.90	3.32	3.73	4.15	4.56	4.98
40	0.69	1.03	1.38	1.72	2.07	2.41	2.76	3.10	3.45	3.79	4.13
50	0.60	0.90	1.19	1.49	1.79	2.09	2.39	2.69	2.98	3.28	3.58
60	0.53	0.80	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2.92	3.18
90	0.41	0.61	0.82	1.02	1.23	1.43	1.63	1.84	2.04	2.25	2.45
120	0.34	0.51	0.68	0.85	1.02	1.19	1.36	1.53	1.70	1.87	2.04
150	0.29	0.44	0.59	0.73	0.88	1.03	1.18	1.32	1.47	1.62	1.76
180	0.26	0.39	0.52	0.65	0.78	0.91	1.04	1.18	1.31	1.44	1.57
240	0.22	0.33	0.43	0.54	0.65	0.76	0.87	0.98	1.08	1.19	1.30
300	0.19	0.28	0.38	0.47	0.56	0.66	0.75	0.85	0.94	1.03	1.13
360	0.17	0.25	0.33	0.42	0.50	0.58	0.67	0.75	0.84	0.92	1.00

Intensity-Duration Design Chart - Example

Table 3-1
RUNOFF COEFFICIENTS FOR URBAN AREAS

Land Use		Runoff Coefficient "C"				
			Soil Type			
NRCS Elements	County Elements	% IMPER.	A	B	C	D
Undisturbed Natural Terrain (Natural)	Permanent Open Space	0*	0.20	0.25	0.30	0.35
Low Density Residential (LDR)	Residential, 1.0 DU/A or less	10	0.27	0.32	0.36	0.41
Low Density Residential (LDR)	Residential, 2.0 DU/A or less	20	0.34	0.38	0.42	0.46
Low Density Residential (LDR)	Residential, 2.9 DU/A or less	25	0.38	0.41	0.45	0.49
Medium Density Residential (MDR)	Residential, 4.3 DU/A or less	30	0.41	0.45	0.48	0.52
Medium Density Residential (MDR)	Residential, 7.3 DU/A or less	40	0.48	0.51	0.54	0.57
Medium Density Residential (MDR)	Residential, 10.9 DU/A or less	45	0.52	0.54	0.57	0.60
Medium Density Residential (MDR)	Residential, 14.5 DU/A or less	50	0.55	0.58	0.60	0.63
High Density Residential (HDR)	Residential, 24.0 DU/A or less	65	0.66	0.67	0.69	0.71
High Density Residential (HDR)	Residential, 43.0 DU/A or less	80	0.76	0.77	0.78	0.79
Commercial/Industrial (N. Com)	Neighborhood Commercial	80	0.76	0.77	0.78	0.79
Commercial/Industrial (G. Com)	General Commercial	85	0.80	0.80	0.81	0.82
Commercial/Industrial (O.P. Com)	Office Professional/Commercial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (Limited I.)	Limited Industrial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (General I.)	General Industrial	95	0.87	0.87	0.87	0.87

*The values associated with 0% impervious may be used for direct calculation of the runoff coefficient as described in Section 3.1.2 (representing the pervious runoff coefficient, Cp, for the soil type), or for areas that will remain undisturbed in perpetuity. Justification must be given that the area will remain natural forever (e.g., the area is located in Cleveland National Forest).

DU/A = dwelling units per acre

NRCS = National Resources Conservation Service

BASIN B-2

FEATURES

Impeller: Cast iron, enclosed, non-clog, dynamically balanced with pump out vanes for mechanical seal protection.

Casing: Cast iron flanged volute type for maximum efficiency. Designed for easy installation on A10-20 slide rail or base elbow rail systems.

Mechanical Seal: SILICON CARBIDE VS. SILICON CARBIDE sealing faces for superior abrasive resistance, stainless steel metal parts, BUNA-N elastomers.

Shaft: Corrosion-resistant, 300 series stainless steel. Threaded design. Locknut on all models to guard against component damage on accidental reverse rotation.

Fasteners: 300 series stainless steel.

Capable of running dry without damage to components.

Designed for continuous operation when fully submerged.

EXTENDED WARRANTY AVAILABLE FOR RESIDENTIAL APPLICATIONS.

WS_BHF Series

Model 3887BHF

SUBMERSIBLE SEWAGE PUMP

GOULDS
WATER TECHNOLOGY
a xylem brand

APPLICATIONS

Specifically designed for the following uses:

- Homes
- Sewage systems
- Dewatering/Effluent
- Water transfer
- Light industrial
- Commercial applications

Anywhere waste or drainage must be disposed of quickly, quietly and efficiently.

SPECIFICATIONS

Pump

- Solids handling capabilities: 2" maximum
- Capacities: up to 220 GPM
- Total heads: up to 81 feet TDH
- Discharge size: 2" NPT threaded companion flange as standard. 3" option available but must be ordered separately. (Order no. A1-3)
- Temperature: 104°F (40°C) continuous
  140°F (60°C) intermittent.

MOTORS

- Fully submerged in high grade turbine oil for lubrication and efficient heat transfer. All ratings are within the working limits of the motor.

Class B insulation on ⅓-1½ HP models.

Class F insulation on 2 HP models.

Single phase (60 Hz):

- Capacitor start motors for maximum starting torque.
- Built-in overload with automatic reset.
- SJTOW or STOW severe duty oil and water resistant power cords.
- ⅓ – 1 HP models have NEMA three prong grounding plugs.
- 1½ HP and larger units have bare lead cord ends.

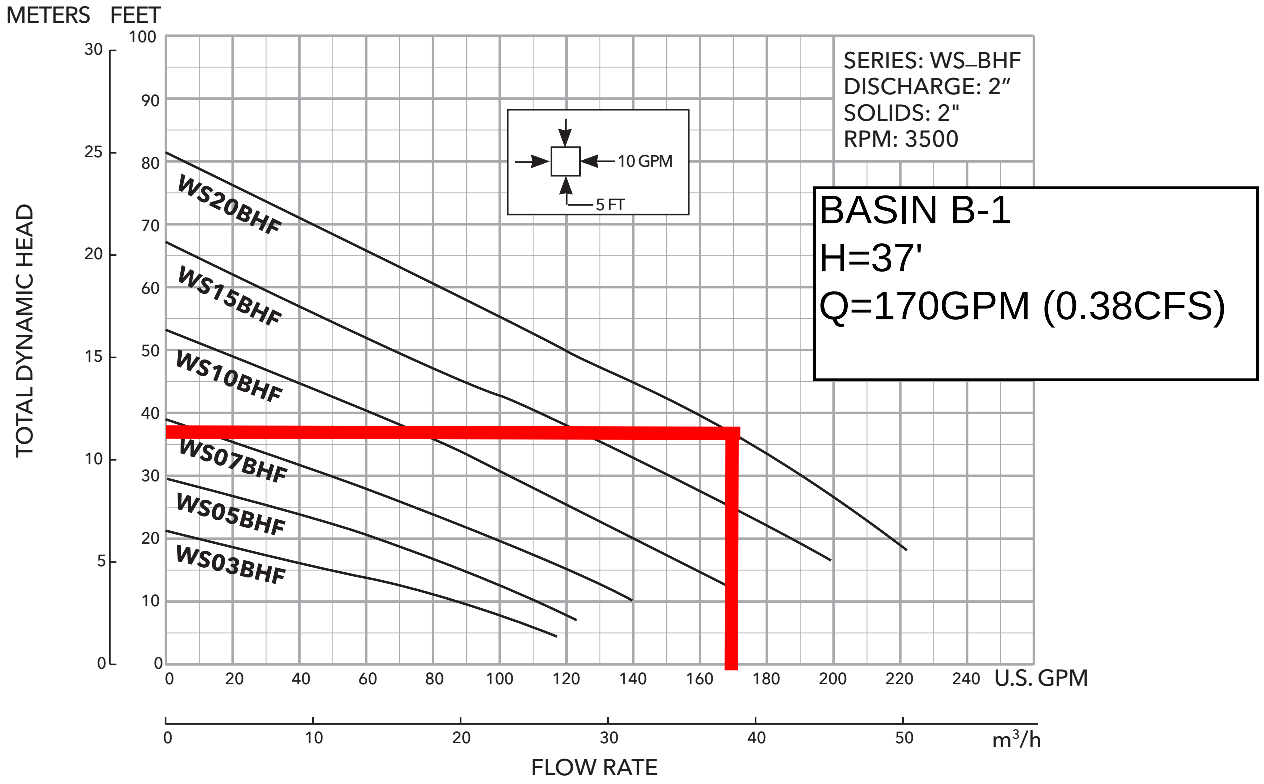
Three phase (60 Hz):

- Class 10 overload protection must be provided in separately ordered starter unit.
- STOW power cords all have bare lead cord ends.

- Bearings: Upper and lower heavy duty ball bearing construction.
- Designed for Continuous Operation: Pump ratings are within the motor manufacturer's recommended working limits, can be operated continuously without damage when fully submerged.
- Power Cable: Severe duty rated, oil and water resistant. Epoxy seal on motor end provides secondary moisture barrier in case of outer jacket damage and to prevent oil wicking. Standard cord is 20'. Optional lengths are available.
- Motor Cover O-ring: Assures positive sealing against contaminants and oil leakage.

AGENCY LISTINGS

Tested to UL 778 and CSA 22.2 108 Standards
By Canadian Standards Association
File #LR38549



PRELIMINARY GEOTECHNICAL EVALUATION
2585 CALLE DEL ORO, COMMUNITY OF LA JOLLA
CITY OF SAN DIEGO, SAN DIEGO COUNTY, CALIFORNIA

FOR

MS. FAYE TASSVIRI
C/O CDGI
1517 SUTTER STREET
SAN DIEGO, CALIFORNIA 92103

W.O. 4971-A-SC JUNE 19, 2007

GeoSoils, Inc.

Geotechnical • Coastal • Geologic • Environmental

5741 Palmer Way • Carlsbad, California 92010 • (760) 438-3155 • FAX (760) 931-0915

June 19, 2007

W.O. 4971-A-SC

Ms. Faye Tassviri
c/o **CDGI**
1517 Sutter Street
San Diego, California 92103

Attention: Mr. Francisco Mendiola

Subject: Preliminary Geotechnical Evaluation, 2585 Calle Del Oro, Community of La Jolla, City of San Diego, San Diego County, California

Dear Mr. Mendiola:

In accordance with your request, GeoSoils, Inc. (GSI) is pleased to present the results of our geotechnical evaluation of the subject site. The purpose of our evaluation was to evaluate the geologic and geotechnical conditions of the site, relative to the proposed development, and to present recommendations for grading and foundation design, and construction for the proposed development.

EXECUTIVE SUMMARY

Based on our experience in the site vicinity, field exploration, geologic and geotechnical engineering analysis, the proposed development appears feasible from a soils engineering and geologic viewpoint, provided that the recommendations presented herein are properly incorporated into the design and construction of the project. The most significant elements of our study are summarized below:

- The site appears to be primarily underlain by formational sedimentary deposits belonging to the Tertiary-age Ardath Shale, with a westward thickening surficial layer of existing fill, overlying the sedimentary bedrock. Bedrock is considered to be suitable for the support of fills and settlement-sensitive improvements; however, existing artificial fill overlying the bedrock is considered to be unsuitable for the support of settlement-sensitive improvements. As such, a deepened foundation system, penetrating surficial fills, and bearing in the underlying sedimentary bedrock, is recommended.

- Our review indicates that regional groundwater should not significantly affect site development. In general, perched groundwater conditions, along zones of contrasting permeabilities, may not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities. Perched groundwater should be anticipated to occur after development, and may require additional mitigation when it manifests itself. This would need to be disclosed to any homeowners and/or other interested/affected parties.

- Based on the absence of a regional groundwater table and the relatively dense condition of sediments underlying the site, the liquefaction potential onsite is considered very low. Other seismic hazards, such as fault rupture, ground lurching, and ground shaking, have a relatively higher potential to occur during an earthquake on one of the nearby faults. However, it should be noted that faults, active or otherwise, do not appear to underlie the site, and the potential risk due to these hazards is no greater than for other residential structures in the general vicinity, based on the available data.

- Based on our experience in the vicinity, a review of the documents referenced in Appendix A, and testing performed in preparation of this report, soils underlying the site are generally clayey, and medium expansive (Uniform Building Code/California Building Code ([UBC/CBC], International Conference of Building Officials [ICBO], 1997 and 2001), and also generally present a negligible sulfate exposure (UBC/CBC [ICBO], 1997 and 2001) to concrete, where tested. Site soils are considered to be corrosive to ferrous materials when saturated. Consultation with a corrosion engineer is recommended.

- Based on the potential for existing fill and topsoil/colluvium to occur beneath and within the proposed new building footprint, proposed grades, and the proximity to offsite structures and/or property lines, a pier and grade beam foundation system, or a combination pier/grade beam - shallow (conventional/post-tension) foundation, founded entirely in suitable bedrock, is recommended. Structural slabs are recommended for slabs-on-grade.

- Adverse geologic structures, to the depths explored, were not encountered. Active faulting was also not encountered onsite.

- An analysis of global slope stability indicates that the existing west facing slope onsite is relatively stable and displays a factor of safety against failure in excess of 1.5 (static), and 1.1 (seismic), under normal conditions of care, maintenance, and rainfall.

- The demolition of the existing structure should not be completed until this report is approved by the controlling authorities for this project and addition.

- Budgetary provisions for appropriate mitigation of the above conditions as well as all ramifications, should be included in project planning.
- The recommendations presented herein should be included in project planning, design and construction.

We appreciate the opportunity to be of service. If you have any questions pertaining to this report, please contact us at (760) 438-3155.

Respectfully submitted,

GeoSoils, Inc.

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PRELIMINARY GEOTECHNICAL EVALUATION
2585 CALLE DE ORO, COMMUNITY OF LA JOLLA
CITY OF SAN DIEGO, SAN DIEGO COUNTY, CALIFORNIA

SCOPE OF SERVICES

The scope of our services has included the following:

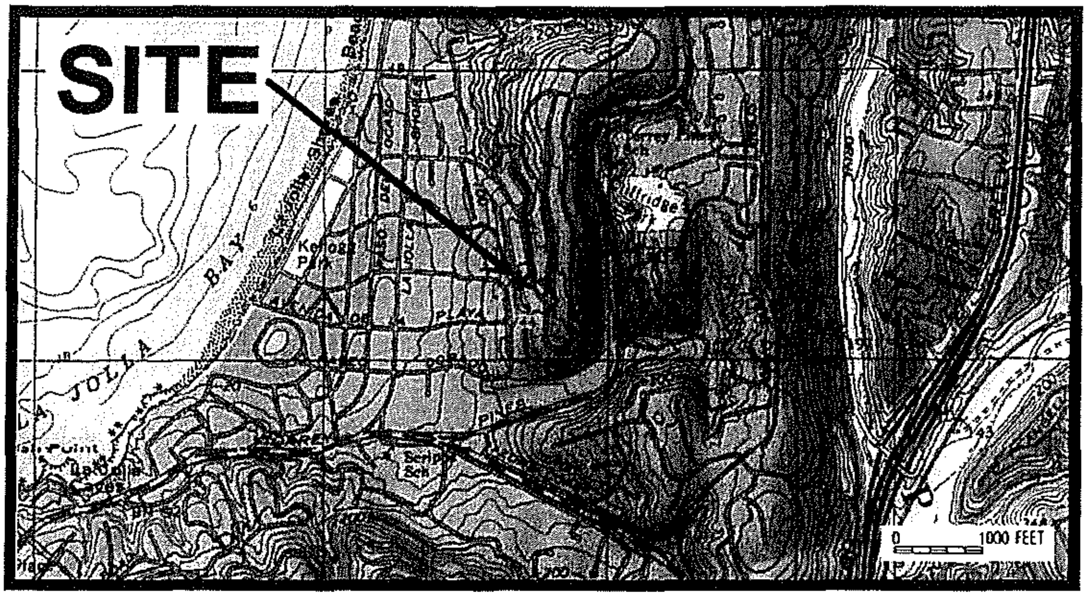
1. Review of readily available soils and geologic data, including soil information contained within a previous geotechnical report for the site (Appendix A).
2. Preparation of a geotechnical map for the site and a geologic cross-section.
3. Subsurface exploration consisting of excavation of two exploratory borings with a drill rig and a hand auger for geotechnical logging and sampling (Appendix B).
4. Evaluation of regional seismicity and seismic hazards (Appendix C).
5. Laboratory testing of representative soil samples collected during our subsurface exploration program.
6. Appropriate engineering and geologic analysis of data collected and preparation of this report. This report does not address existing or potential environmental concerns relative to the subject property.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

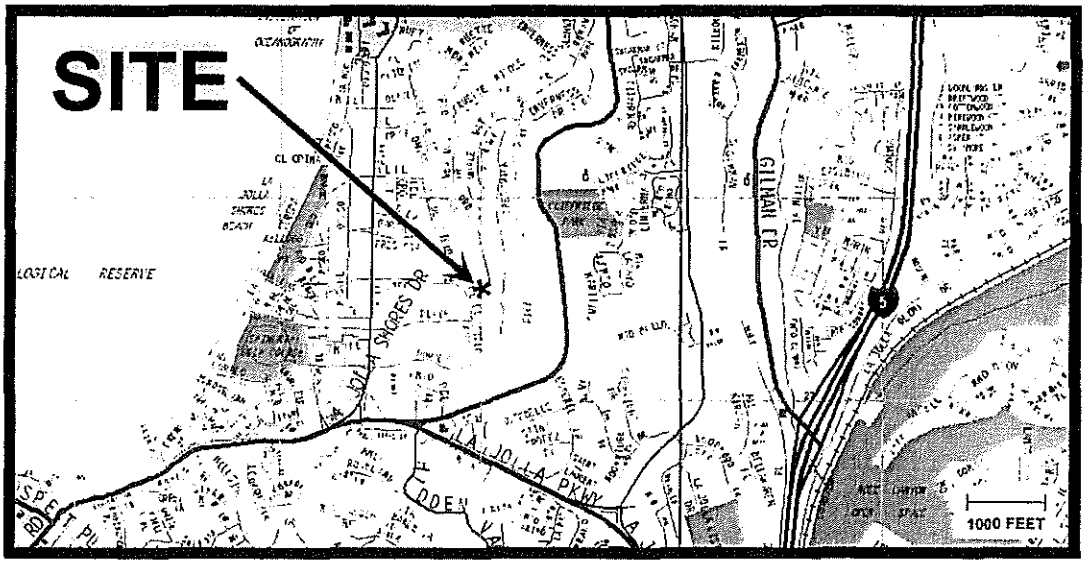
The subject site is located on the west side of Calle De Oro, in the Community of La Jolla, City of San Diego, San Diego County, California (see Site Location Map, Figure 1). The property is bounded on the remaining sides by single-family residential development.

Topographically, the site consists of a relatively flat lying building pad near street level, with an existing slope descending from the rear (west) of the pad toward offsite properties. This slope is on the order of 40 to 50 feet in height, with an average gradient on the order of 1.8:1 (horizontal to vertical [h:v]). Existing improvements onsite consist of a single-family residential structure with an attached garage, swimming pool, and associated exterior improvements (i.e., flatwork, landscaping, etc.). Drainage appears to be directed offsite to the west.

It is our understanding that the existing residence is planned to undergo a major remodel, resulting in the removal of a majority of the existing improvements onsite. It is also our understanding that the existing pad grades will not significantly change toward the front (east) of the pad, but will be lowered on the order of 5 to 7 feet within the western portion of the pad. It is anticipated that the planned remodel will use continuous footings and slab-on-grade, or raised wood floors. Building loads are assumed to be typical for this relatively lightly loaded structure. Sewage disposal is anticipated to be tied into the regional system.



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S. La Jolla Qudrangle (dated 1996, current 1996) and La Jolla OEW Quadrangle (dated 1975, current 1975), California–San Diego Co., 7.5-Minute.



Base Map: The Thomas Guide, San Diego Co. Street Guide and Directory, 2005 Edition, by Thomas Bros. Maps, pages 1227 and 1228.

LOCATION AND SCALES APPROXIMATE

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W.O. 4971-A-SC

SITE LOCATION MAP

Figure 1

SITE BACKGROUND

A previous geotechnical study was performed by Christian Wheeler Engineering (CWE, 2002). Site work completed in preparation of CWE (2002) consisted of subsurface exploration with both a large and small diameter drill rigs. Samples obtained from these borings were tested, with the results of testing included in CWE (2002). The approximate location of borings specifically referred to in this study are shown on Plate 1.

FIELD STUDIES

Field studies conducted by GSI consisted of geologic mapping of the site and the excavation of two exploratory borings with a a small diameter drill rig and a hand auger, in order to supplement the existing data base presented in CWE (2002). The borings were logged by an engineering geologist from our firm who collected representative bulk and undisturbed samples from the borings for appropriate laboratory testing. Boring Logs are presented in Appendix B. The locations of the borings are presented on Plate 1 (Geotechnical Map). Plate 1 was adapted from the site plan prepared for this site by CDGI (2007).

REGIONAL GEOLOGY

The subject property is located within a prominent natural geomorphic province in southwestern California known as the Peninsular Ranges. It is characterized by steep, elongated mountain ranges and valleys that trend northwesterly. The mountain ranges are underlain by basement rocks consisting of pre-Cretaceous metasedimentary rocks, Jurassic metavolcanic rocks, and Cretaceous plutonic rocks of the southern California batholith.

In the San Diego County region, deposition occurred during the Cretaceous period and Cenozoic era in the continental margin of a forearc basin. Sediments, derived from Cretaceous-age plutonic rocks and Jurassic-age volcanic rocks, were deposited into the narrow, steep, coastal plain and continental margin of the basin during the Tertiary period. These rocks have been uplifted, tilted, faulted, eroded, and deeply incised. During early Pleistocene time, a broad coastal plain was developed from the deposition of marine terrace deposits. During mid to late Pleistocene time, this plain was uplifted, eroded, and incised. Alluvial deposits have since filled the lower valleys, and young marine sediments are currently being deposited/eroded within coastal and beach areas. Based on our review, the site appears to be underlain with Tertiary-age sedimentary bedrock, belonging to the Ardath Shale.

EARTH MATERIALS

Earth materials encountered on the site consist of undocumented artificial fill and sedimentary bedrock belonging to the Tertiary-age Ardath Shale. The limits for these earth materials are indicated on Plate 1 (Geotechnical Map). Subsurface conditions are shown in cross-section on Plate 2.

Topsoil/Colluvium (Not Mapped)

Topsoil/colluvium occurs locally as a thin, discontinuous surficial layer of soil throughout the site. Where observed, these materials consist of a brown, lean clay, and are on the order of 1 to 3 feet in thickness. Based on the available data, these soils are considered potentially compressible in their existing state and will require removal and recompaction, if settlement-sensitive improvements are proposed within their influence. Alternatively, a deepened foundation system (with a structural slab), penetrating these soils and embedded into the underlying bedrock, may be constructed.

Artificial Fill - Undocumented (Map Symbol - Afu)

Undocumented artificial fill generally occurs as a westward thickening wedge of soil with the thickest fills located near the western edge of the existing building pad (i.e. near the top of the existing west facing slope). Where observed, existing fills appear to attain maximum thickness on the order of 15 to 20 feet near the top of the existing west facing fill slope (CWE, 2002). Artificial fill is comprised of a brown, lean clay. These soils were noted to be moist to very moist, and medium stiff to hard (CWE, 2002). The relative compaction of the fill encountered within the CWE borings was evaluated to range from approximately 80 percent to in excess of 90 percent relative compaction, with the denser fills occurring at depths greater than approximately 5 feet to existing pad grade. Based on the available data, existing fill soils are considered potentially compressible in their existing state, and will require removal and recompaction if settlement-sensitive improvements are proposed within their influence. Alternatively, a deepened foundation system (with a structural slab), penetrating existing fills and embedded into the underlying bedrock, may be constructed.

Ardath Shale (Map Symbol - Ta)

Based on our review and field exploration, existing fills are underlain by sedimentary bedrock belonging to the Tertiary-age Ardath Shale. These formational materials encountered onsite consist of light brown silty to fine sandy claystone. Where observed, bedrock is typically moist and hard.

From a structural viewpoint, the Ardath Shale in the vicinity is a thinly bedded formation, with bedding generally dipping on the order of 3 to 5 degrees to the northeast (i.e., into slope). High angle fractures, dipping on the order of 50 to 60 degrees to the northwest, were also observed. These fractures are not pervasive, and occasionally infilled with gypsum. Fractures were generally not observed below depths of 27 to 30 feet (this study and CWE, 2002). It should be noted that the findings of CWE (2002) and this study are generally consistent with regional mapping by Kennedy (1975) in this area. These formational soils are considered suitable for the support of engineered fills and settlement-sensitive improvements.

GROUNDWATER

Regional subsurface water was not encountered within the property during field work performed in preparation of this report. Regional groundwater is not anticipated to adversely affect site development, provided that the recommendations contained in this report are incorporated into final design and construction and that prudent surface and subsurface drainage practices are incorporated into the construction plans. These observations reflect site conditions at the time of our investigation and do not preclude future changes in local groundwater conditions from excessive irrigation, precipitation, or that were not obvious, at the time of our investigation.

Seeps, springs, or other indications of a high groundwater level were not noted on the subject property during the time of our field investigation. However, seepage may occur locally (due to heavy precipitation or irrigation) in areas where fill soils overlie silty or clayey soils. Such soils may be encountered in the earth units that exist onsite, and should be anticipated both during grading and after development. This potential will need to be disclosed to the homeowner and/or any other interested/affected parties.

Perched groundwater conditions along fill/bedrock contacts and along zones of contrasting permeabilities should not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities, and should be anticipated. Should perched groundwater conditions develop, this office could assess the affected area(s) and provide the appropriate recommendations to mitigate the observed groundwater conditions. This potential should be disclosed to the homeowner and/or any other interested/affected parties.

FAULTING AND REGIONAL SEISMICITY

San Andreas Transform Fault System

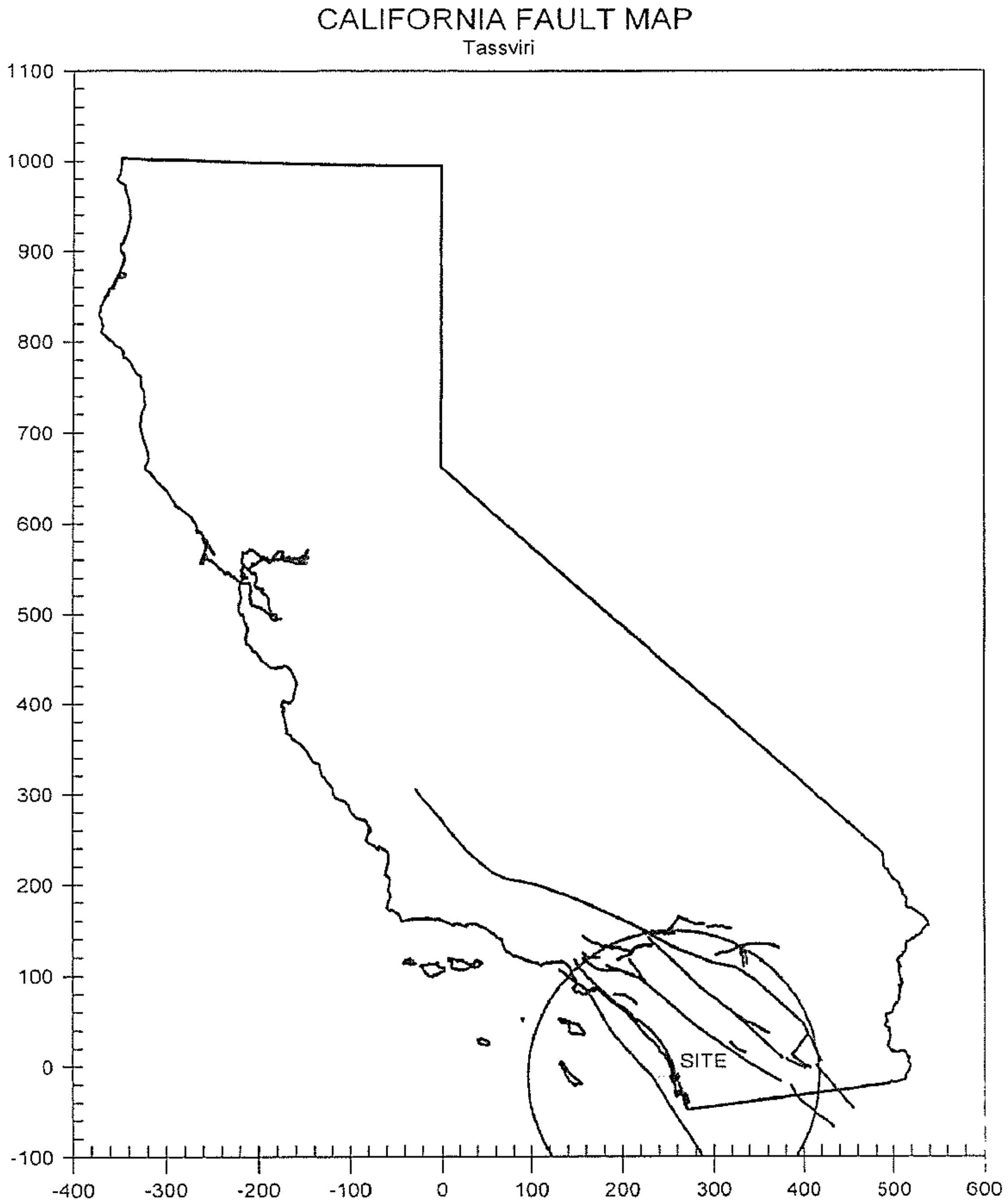
The San Andreas transform-fault system is a family of right-lateral faults that evolved along the continental margin of western North America since middle Miocene time in response to interactions between the North American Plate and various oceanic plates to the west. Depending on the plate size, geometry, and boundary conditions, this motion produced either rotation and translation (e.g., the western Transverse Ranges), transtensional rifting (e.g., the outer borderland), or partitioning of strain into nearly pure strike-slip motion (e.g., Baja California). As the transform system evolved in a simple shear environment (i.e., only the Pacific Plate is moving obliquely), a geometric relationship developed among fault structures, with the San Andreas fault zone becoming the principal displacement zone. More northerly striking faults evolved (synthetic shears) and east-westerly faults evolved (antithetic shears). The San Andreas fault zone (the principal displacement zone), and the northerly trending faults that developed showed right-lateral slip, whereas, the east-westerly trending faults that developed originally showed left-lateral slip (Sylvester, 1988). A similar scenario may have initiated the easterly trending normal faults exposed in the coastal bluffs and nearby areas, from Point Loma to northeast of La Jolla. Alternatively, these easterly trending faults may be a result of local extensional stress in a northwest-southeast oriented direction.

As summarized by Matti, et al. (1992a and 1992b), in central California, displacement has occurred mainly along the San Andreas proper. In southern California, however, the total displacement has been taken up by several discreet fault strands, including the San Andreas, San Jacinto, Punchbowl, San Gabriel, and Banning faults, as well as other structures (Matti and Morton, 1993), with some displacement being partitioned to the Elsinore, Newport-Inglewood - Rose Canyon, and Coronado Bank faults, among others.

The California Continental Borderland is a complex part of the continental transform fault boundary between the Pacific and North American tectonic plates (Legg and Kennedy, 1991). The region is underlain by numerous Cenozoic faults that are subparallel to the San Andreas fault. The Newport-Inglewood - Rose Canyon fault zone is considered a part of, or aligned with, this zone. Although, in general, these faults are mostly right-slip in character, conforming with the relative plate motion in the region, segments of the offshore fault zones show local convergence or extension associated with bends. In addition, regional variability in partitioning of the plate boundary strain across and among these faults also results in many fault segment showing oblique movement. These faults have the potential to generate uplift or subsidence during a major offshore earthquake which could result in generation of a tsunami, such as was observed in 1927 offshore of Lompoc, in Central California. The relative location of major fault systems in the region are shown on Figure 2 (California Fault Map).

The Newport-Inglewood - Rose Canyon Fault Zone

As summarized by Fischer and Mills (1991), the Newport-Inglewood - Rose Canyon fault zone (NI-RC) trends southeast from the east-west trending Santa Monica fault zone in the north, through San Diego Bay in the south and is considered to be one continuous fault zone. The southern Rose Canyon fault zone (RCFZ) may connect to the Pescadero fault near the International Border and become part of the Agua Blanca system in Baja California. The northern Newport-Inglewood fault zone (NIFZ) in the Los Angeles basin is a narrow belt of discontinuous, dominantly left-stepping, en echelon faults and folds that is the result of movement along a major through-going, right-slip fault in basement rocks. The southern onshore NIFZ and the NI-RC zone are in general less complex zones of linear dominantly left-stepping shears.



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CALIFORNIA FAULT MAP

Figure 2

W.O. 4971-B-SC	DATE 06/97	SCALE NTS

The site is located near the RCFZ (Fischer and Mills, 1991), approximately 2 miles northeast of the southern RCFZ (San Diego segment). The major restraining bend of the RCFZ, at the northern end of the San Diego segment has resulted in the uplift of the Point Loma-La Jolla block, and a pull-apart zone associated with San Diego Bay and associated grabens (downtown graben, etc.). To the south, the releasing bend of the zone across San Diego Bay, and a series of individual fault splays characterizes the boundary between the San Diego and Silver Strand segments. The fault zone continues from San Diego Bay northward into Mission Hills and Old Town, where the Silver Strand, Coronado, Spanish Bight faults converge into the Old Town fault, which then merges into the Mission Bay fault north of Old Town.

The site is located approximately 2 miles (3.2 km) southwest of the known "active" trace of the RCFZ (Kennedy, 1975; Treiman, 1993; and Leighton and Associates, 1983), as discussed above. This zone consists of a continuous, northwest trending, broad zone of right lateral oblique slip faults. Fairly recent studies (Lindvall and Rockwell, 1989) have indicated Holocene activity along several isolated strands of the RCFZ.

Additional studies performed for the Police Administration and Technical Center in downtown San Diego have also indicated Holocene activity as well. As a result of these studies, the State of California has classified a portion of the fault between Mission Bay and La Jolla Cove, and in downtown San Diego, as active (Hart and Bryant, 1997).

SEISMIC HAZARDS

There are a number of faults in the southern California area that are considered active and would have an effect on the site in the form of ground shaking, should they be the source of an earthquake. These include, but are not limited to: the San Andreas fault, the San Jacinto fault, the Elsinore fault, the Coronado Bank fault zone, and the NI-RC fault zone. The location of these and other major faults relative to the site are indicated on Figure 2. The possibility of ground acceleration or shaking at the site may be considered as approximately similar to the southern California region as a whole. The following table lists the major faults and fault zones in southern California that could have a significant effect on the site should they experience significant activity.

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE - MILES (KM)
Rose Canyon	2.0 (3.2)
Coronado Bank - Agua Blanca	13.2 (21.2)
Newport-Inglewood-Offshore	23.4 (37.6)
Elsinore - Julian	37.1 (59.7)
Elsinore - Temecula	38.6 (62.2)

In addition to seismic "shaking," the following list includes other seismic related hazards that have been considered during the evaluation of the site. The hazards listed are considered negligible and/or completely mitigated as a result of site location, soil characteristics, and typical site development procedures:

- Dynamic Settlement
- Surface Fault Rupture
- Ground Lurching or Shallow Ground Rupture
- Tsunami
- Liquefaction
- Seiche

It is important to keep in perspective that in the event of an earthquake occurring on any of the nearby major faults, strong ground shaking would occur in the subject site's general area. Potential damage to any structure(s) would likely be greatest from the vibrations and impelling force caused by the inertia of a structure's mass than from those induced by the hazards considered above. This potential would be no greater than that for other existing structures, and improvements in the immediate vicinity.

REGIONAL SEISMICITY

The acceleration-attenuation relations of Sadigh, et al. (1997) Horizontal Soil, Bozorgnia, Campbell, and Niazi (1999) Horizontal-Soft Rock-Correlation, and Campbell and Bozorgnia (1997 Rev.) Horizontal-Soil have been incorporated into EQFAULT (Blake, 2000a). For this study, peak horizontal ground accelerations anticipated at the site were determined based on the random mean plus 1 - sigma attenuation curve and mean attenuation curve developed by Sadigh, et. al. (1997), Bozorgnia, Campbell, and Niazi (1999), and Campbell and Bozorgnia (1997). EQFAULT is a computer program by Thomas F. Blake (2000a), which performs deterministic seismic hazard analyses using up to 150 digitized California faults as earthquake sources. Printouts presenting some of the following results are presented in Appendix C.

The program estimates the closest distance between each fault and a given site. If a fault is found to be within a user-selected radius, the program estimates peak horizontal ground acceleration that may occur at the site from an upper bound ("maximum credible") earthquake on that fault. Site acceleration (g) is computed by one of many user-selected acceleration-attenuation relations that are contained in EQFAULT. Based on the EQFAULT program, peak horizontal ground accelerations from an upper bound event at the site may be on the order of 0.83 g to 0.94 g.

Historical site seismicity was evaluated with the acceleration-attenuation relations of Campbell and Bozorgnia (1997 Revised), and Bozorgnia, Campbell, and Niazi (1999) and the computer program EQSEARCH (Blake, 2000b). This program performs a search of the historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-mile radius, between the years 1800 to June 2006. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have effected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 to June 2006, was on the order of 0.38 g. Site specific probability of exceeding various peak horizontal ground accelerations and a seismic recurrence curve are also estimated/generated from the historical data.

A probabilistic seismic hazards analyses was performed using FRISKSP (Blake, 2000c) which models earthquake sources as three-dimensional planes and evaluates the site specific probabilities of exceedance for given peak acceleration levels or pseudo-relative velocity levels. Based on a review of these data, and considering the relative seismic activity of the southern California region, a peak horizontal ground acceleration on the order of 0.33 g to 0.40 g was calculated. This range was chosen as it corresponds to a 10 percent probability of exceedance in 50 years (or a 475-year return period). The higher acceleration should be used by the structural consultant and architect in the evaluation of the site improvements.

Experience has shown that wood-frame structures designed in accordance with the Uniform Building Code/California Building Code ([UBC/CBC], International Conference of Building Officials [ICBO], 2001 and 1997), tend to mitigate earthquake effects. Earthquake effects may include lurching and/or localized ground cracking. This effect is similar to other portions of southern California.

Ground lurching or shallow ground rupture due to shaking could occur in the site area from an earthquake originating on other nearby faults. Such lurching could possibly cause cracking of paved areas, with limited damage to structures. Based on the site conditions, Chapter 16 of the UBC (ICBO, 1997), the following seismic parameters are provided.

Seismic zone (per Figure 16-2*)	4
Seismic Zone Factor (per Table 16-I*)	0.40
Soil Profile Type (per Table 16-J*)	S_C
Seismic Coefficient C_a (per Table 16-Q*)	0.40 N_a
Seismic Coefficient C_v (per Table 16-R*)	0.56 N_v
Near Source Factor N_a (per Table 16-S*)	1.18
Near Source Factor N_v (per Table 16-T*)	1.44
Seismic Source Type (per Table 16-U*)	B
Distance to Seismic Source	2.0 mi/3.2 km
Upper Bound Earthquake (Rose Canyon)	M_w 7.2
* Figure and table references from Chapter 16 of the UBC (ICBO, 1997).	

MASS WASTING

The site and vicinity have been categorized by the City (Leighton and Associates, 1983) as an area of "potential slope instability," but underlain with formations with "neutral, or favorable" geologic structure." The "Risk Zone" for this area is also identified as "low" with "no special hazards identified nearby." A review of available geologic/topographic maps (Kennedy, 1975) and aerial photographs (USDA, 1953) did not indicate the presence of any geomorphic features possibly indicative of a regional landslide, or other "mass wasting" types of deposits. Bedrock structure generally appears to be inclined into slope, and is not considered adverse with respect to existing, or any planned slope onsite, based on the available data, and to the depths explored.

SLOPE STABILITY

The existing west facing slope, located below the existing building pad, was evaluated for slope stability. Using the modified Bishop's Method, and soil parameters presented herein, our stability analysis indicates that this slope is grossly and surficially stable, assuming proper construction, maintenance, and normal rainfall. Our slope stability analysis is presented in Appendix D.

LABORATORY TESTING

Laboratory tests were performed on representative samples of site earth materials in order to evaluate their physical characteristics. Additionally, laboratory testing performed in preparation of CWE (2002) was also reviewed and made a part of this report. Test procedures used and results obtained from CWE (2002) and this study, are presented below.

Classification

Soils were classified visually according to the Unified Soils Classification System (USCS). The soil classifications are shown on the Exploratory Excavation Logs (see Appendix B).

Moisture-Density Relations

The field density and field moisture content were determined for the major soil types encountered in the borings. Results of this testing are presented on the Boring Logs (Appendix B).

Laboratory Standard

The maximum density and optimum moisture content was evaluated for a representative sample of onsite soil in general accordance with ASTM D-1557. The moisture-density relationships obtained for this soil is shown in the following table:

LOCATION	SOIL TYPE	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
B-1 @ 4'-9' (CWE, 2002)	SILTY CLAY	115.1	12.1

Expansion Potential

Expansion testing was performed on representative samples of site soil in general accordance with the 1997 UBC Standard 18-2. Additional testing should be conducted during site re-grading. The results of expansion testing are presented in the following table.

LOCATION	EXPANSION INDEX	EXPANSION POTENTIAL
Boring B-1 @ 5'	62	Medium
Boring B-1 @ 20' - 24'	70	Medium
Boring CWE-1 @ 4'-9' (CWE, 2002)	49	Low

Atterberg Limits

Tests were performed to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D-4318. The test results are presented below:

LOCATION	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Boring B-1 @ 5'	45	19	26
Boring CWE-1 @ 4'-9' (CWE, 2002)	42	21	21
Boring CWE-1 @ 20' (CWE, 2002)	47	28	19

Shear Testing

Shear testing was performed on two undisturbed samples in formational materials in general accordance with ASTM Test Method D-3080 in a Direct Shear Machine of the strain control type. The shear test results from this study, as well as CWE (2002) are presented below:

SAMPLE LOCATION	PRIMARY		RESIDUAL	
	COHESION (psf)	FRICTION ANGLE (degrees)	COHESION (psf)	FRICTION ANGLE (degrees)
Boring B-1 @ 10' (undisturbed bedrock)	2200	23	853	32
Boring B-1 @ 25' (undisturbed bedrock)	1800	25	875	36
Boring CWE-1 @ 4'-9' (undisturbed fill)	450	28	--	--
Boring CWE-4 @ 30' (undisturbed bedrock)	500	37	--	--

Sieve Analysis

The gradation of selected soil samples was evaluated in general accordance with ASTM D-422. The results of the sieve analyses are presented in the following table.

SIEVE SIZE	BORING B-1 @ 5' (#200 sieve only)	BORING CWE-1 @ 4'-9'	BORING CWE-1 @ 9'-15'
#4	--	100	100
#8	--	99	100
#16	--	98	99
#30	--	97	98
#50	--	96	98
#100	--	94	97
#200	84	89	92
0.05 mm	--	77	77
0.005 mm	--	37	25
0.001 mm	--	3	0
Classification	CL	CL	CL

Soluble Sulfates/Corrosion

A representative sample of site material was analyzed for preliminary corrosion/soluble sulfate potential. The testing included evaluation of pH, soluble sulfates, and saturated resistivity. Test results indicate that the soil presents a severe sulfate exposure to concrete, in accordance with Table 19-A-4 of the UBC (1997 edition), and is corrosive to ferrous metals based on saturated resistivity. Site soils are considered to be relatively neutral (pH = 7.6). A corrosion specialist should be consulted for the appropriate mitigation recommendations, where site soils will come into contact with piping, foundations, metals, etc. Based on the sulfate conditions, Type V cement (per Table 19-A-4 of the UBC [ICBO, 1997]) is recommended. Test results are shown on Figure 3.

PRELIMINARY CONCLUSIONS

Based upon our site reconnaissance, subsurface exploration, and laboratory test results, it is our opinion that the subject site appears suitable for the proposed remodel/additional construction. Based on our evaluation, the following general conditions are noted:

- The existing residential structure is proposed to be remodeled, including partial removal of portions of the structure. The building pad will likely expose a cut/fill transition during site work. Existing fills are unsuitable, and potentially compressible. New foundation systems should be embedded below any existing fill, into the underlying formational material. The existing residence should not be demolished until this report and addition are approved by the governing authorities.

- It is our understanding that the existing pad grade will generally remain the same as the existing grade. This would generally result in a pad exposing formational material at grade on the east side, and undocumented fill, on the west side. Based on our subsurface data, the thickness of undocumented fill could vary up to 15 to 20 feet in thickness along the top of the existing slope.

- As such, a pier and grade beam foundation system appears warranted for the support of the new structures in areas underlain with existing fill. Those portions of the building footprint underlain with suitable sedimentary bedrock may use a structural slab with deepened footings integrated into the pier and grade beam design, or the entire foundation system may use a pier and grade beam design.

- The foundation area for the proposed swimming pool and associated structures may be supported by a deep foundation, as noted above.

The following recommendations consider these, as well as other aspect of site design and construction, and should be incorporated into the construction plans and details.

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Table 1 - Laboratory Tests on Soil Samples

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27-Feb-06

Sample ID			B-1 @ 5'
Resistivity		Units	
as-received		ohm-cm	18,000
saturated		ohm-cm	1,100
pH			7.6
Electrical Conductivity		mS/cm	2.67
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	5,371
magnesium	Mg^{2+}	mg/kg	129
sodium	Na^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	113
chloride	Cl^{1-}	mg/kg	35
sulfate	SO_4^{2-}	mg/kg	12,876
Other Tests			
ammonium	NH_4^{1+}	mg/kg	na
nitrate	NO_3^{1-}	mg/kg	na
sulfide	S^{2-}	qual	na
Redox		mV	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

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SOLUBLE SULFATES/CORROSION TEST RESULTS

Figure 3

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EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

All grading should conform to the guidelines presented in Appendix Chapter A33 of the UBC (ICBO, 1997), the requirements of the City, and the Grading Guidelines presented in Appendix E, except where specifically superceded in the text of this report. Prior to grading, a GSI representative should be present at the preconstruction meeting to provide additional grading guidelines, if needed, and review the earthwork schedule.

During earthwork construction all site preparation and the general grading procedures of the contractor should be observed and the fill selectively tested by a representative(s) of GSI. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and if warranted, modified and/or additional recommendations will be offered. All applicable requirements of local and national construction and general industry safety orders, the Occupational Safety and Health Act (OSHA), and the Construction Safety Act should be met.

Site Preparation

Debris, vegetation, asphaltic concrete, construction debris, and other deleterious material should be removed from the building area prior to the start of construction. Sloping areas to receive fill should be properly benched in accordance with current industry standards of practice and guidelines specified in the 1997 UBC.

Removals (Unsuitable Surficial Materials)

It should be noted that using a pier/grade beam foundation with a structural slab does not require this type of remedial grading for the support of the proposed residence. If a deep foundation is utilized, pad grade should be proof rolled and compacted to at least 90 percent relative compaction prior to foundation construction in the upper 2 feet. Other settlement-sensitive improvements may also require deep foundations.

Fill Placement

Subsequent to ground preparation, onsite soils may be placed in thin (±6-inch) lifts, cleaned of vegetation and debris, brought to a least optimum moisture content, and compacted to achieve a minimum relative compaction of 90 percent. If soil importation is planned, a sample of the soil import should be evaluated by this office prior to importing, in order to assure compatibility with the onsite site soils and the recommendations presented in this report. Import soils for a fill cap should be low expansive (E.I. less than 50). The use of subdrains at the bottom of the fill cap may be necessary, and subsequently recommended based on compatibility with onsite soils.

Temporary Construction Slopes

Temporary cuts for wall construction should be constructed at a gradient of ½:1 or flatter for slopes exposing sedimentary bedrock to a maximum height of 20 feet, per CAL-OSHA for Type A soils. Temporary cuts should be constructed at a gradient of 1:1 (h:v), or flatter, for slopes exposing existing fill, per CAL-OSHA for Type B soils. Construction materials and/or stockpiled soil should not be stored within 5 feet of the top of any temporary slope. Temporary/permanent provisions should be made to direct any potential runoff away from the top of temporary slopes. Shoring may be required. Temporary slopes should be evaluated during construction by the geotechnical engineer for any comments, or revisions to this recommendation. Removals and/or temporary cuts should be made with sufficient space to allow for subdrains and/or wall back drains as recommended in this report.

PRELIMINARY FOUNDATION DESIGN

General

In the event that the information concerning the proposed development plan is not correct, or any changes in the design, location, or loading conditions of the proposed structure are made, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report are modified or approved in writing by this office.

The information and recommendations presented in this section are not meant to supercede design by the project structural engineer or civil engineer specializing in structural design. Upon request, GSI could provide additional input/consultation regarding soil parameters, as related to foundation design.

The following foundation design is based on footings and slabs bearing on suitable bedrock. Areas exposing existing fill should use pier and grade beam foundations for support. Soils onsite are generally considered to be medium expansive (E.I. range of 60 to 70), and display a plasticity index of 26. In addition to the minimum criteria presented herein, the structural engineer should design the foundation in accordance with the minimum criteria presented in Chapter 18 of the UBC (ICBO, 1997) for expansive soil conditions.

Foundation Design

1. The foundation systems should be designed and constructed in accordance with guidelines presented in the latest adopted edition of the UBC. All new foundations should be embedded entirely into properly compacted fill or into suitable sedimentary bedrock.

2. An allowable bearing value of 2,000 pounds per square foot (psf) may be used for design of footings that maintain a minimum width of 12 inches and a minimum depth of 18 inches, and founded in suitable formational material. This value may be increased by 20 percent for each additional 12 inches in depth to a maximum value of 3,000 psf. In addition, this value may be increased by one-third when considering short duration seismic or wind loads. Isolated pad footings should have a minimum dimension of at least 24 inches square and minimum depth of 24 inches, and be connected in two directions.

3. Passive earth pressure may be computed as an equivalent fluid having a density of 350 pounds per cubic foot (pcf), with a maximum earth pressure of 3,000 psf. Lateral passive pressures for shallow foundations within UBC setback zones should be reduced following a review by the geotechnical engineer.

4. An allowable coefficient of friction between soil and concrete of 0.35 may be used with the dead load forces.

5. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

6. For preliminary design purposes, foundations should be designed to minimally accommodate a differential settlement of up to 1 inch in a 40-foot span (angular distortion of 1/480. As grading plans become available, and based on the as-built configuration of the site, this value may be revised. Differential settlements between new/existing construction may exhibit higher gradients if the recommendations in this report are not followed in the planning, design, and construction. A construction joint to allow relative movement in this area is recommended.

FOUNDATION CONSTRUCTION

General

The following foundation construction recommendations are presented as a minimum criteria from a soils engineering viewpoint. Our recommendations for a conventional, shallow foundation system are provided for bearing soils consisting of suitable bedrock (formational) materials.

Recommendations by the project's design/structural engineer or architect, which may exceed the soils engineer's recommendations, should take precedence over the following minimum requirements. Final foundation design will be provided based on the expansion potential of the near surface soils encountered at the conclusion of grading.

1. Interior and exterior footings should be founded at a minimum depth of 18 inches below the lowest adjacent ground surface for one-story and two-story floor loads into suitable formational soil. Footing widths should be per the UBC (ICBO, 1997). Isolated interior or exterior footings should be founded at a minimum depth of 24 inches below the lowest adjacent ground surface into suitable formational soil. All footings should have two No. 4 reinforcing bars placed at the top and two No. 4 reinforcing bars placed at the bottom of the footing.

2. A grade beam, reinforced as above, and at least 12 inches square, should be provided across the garage, or other large entrances. The base of the reinforced grade beam should be at the same elevation as the adjoining footings.

3. Concrete slabs bearing on suitable formational soil in residential and garage areas should be a minimum of 5 inches thick, and underlain with a vapor retarder consisting of a minimum of 10-mil, polyvinyl-chloride membrane, with all laps sealed, per the UBC (ICBO, 1997). This membrane should be covered with a minimum of 2 inches of sand to aid in uniform curing of the concrete. Pea gravel, totaling 3 inches in thickness, should be placed directly on grade and below the membrane. Additional considerations regarding the transmission of water vapor are presented in a later section of this report.

4. Concrete slabs, including garage slabs, should be minimally reinforced with No. 3 reinforcement bars placed on 18-inch centers, in two horizontally perpendicular directions (i.e., long axis and short axis). All slab reinforcement should be supported to ensure proper mid-slab height positioning during placement of the concrete. "Hooking" of reinforcement is not an acceptable method of positioning.

5. Garage slabs should be poured separately from the residence footings and be quartered with expansion joints or saw cuts. A positive separation from the footings should be maintained with expansion joint material to permit relative movement.

6. Presaturation is not recommended for these soil conditions. However, the moisture content of the subgrade soils should be equal to or greater than optimum moisture to a depth of 12 inches below the adjacent ground grade in the slab areas.

7. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction 90 percent of the laboratory standard, whether it is to be placed inside the foundation perimeter or in the yard/right-of-way areas. This material must not alter positive drainage patterns that direct drainage away from the structural areas and toward the street.

8. Foundations near the top of slope should be deepened to conform to the latest edition of the UBC (ICBO, 1997) and provide a minimum 7-foot horizontal distance

from the slope face. Rigid block wall designs located along the top of slope should be reviewed by a soils engineer.

Slope Setback Considerations for Footings

Footings should maintain a horizontal distance, X, between any adjacent descending slope face and the bottom outer edge of the footing. The horizontal distance, X, may be calculated by using X = H/3, where "H" is the height of the slope. X should not be less than 7 feet, nor need not be greater than 40 feet. X may be maintained by deepening the footings.

DRILLED PIER FOUNDATIONS

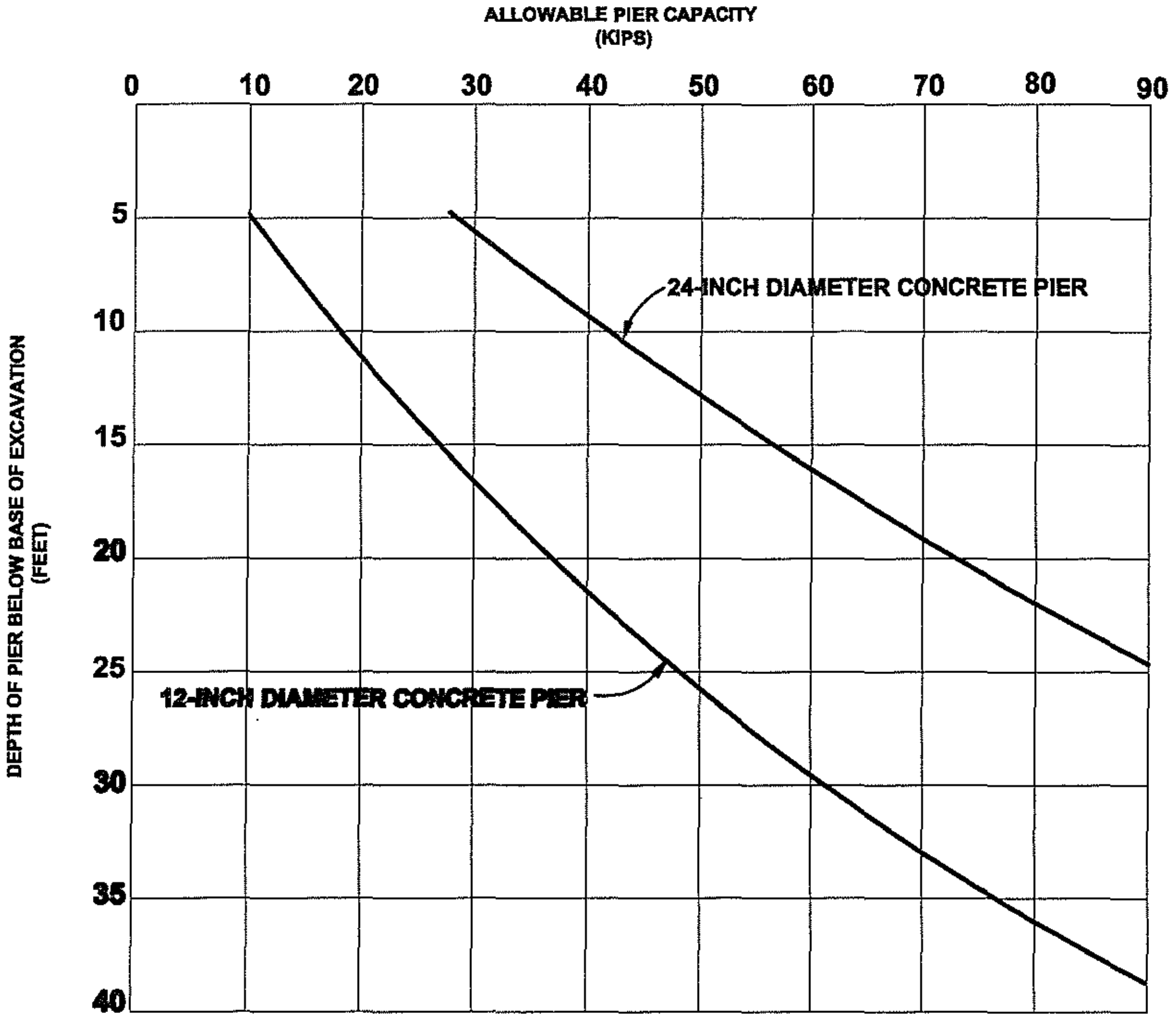
The proposed structures, underlain by left-in-place undocumented artificial fill, may be supported by a drilled, cast-in-place, concrete pier and grade beam system. All drilled piers should extend a minimum of 7 feet into competent formational materials and a minimum of 5 feel into unweathered Ardath Shale. Actual pier embedment should be finalized by the project's structural engineer based on the pier capacity chart (see Figure 4), and the structural capacity of the pier(s) used. The structural strength of the piers should be checked by the structural engineer or civil engineer specializing in structural analysis. Pier holes should be drilled straight and plumb. Locations (both plan and elevation) and plumbness should be the contractors responsibility.

The grade beam should be at a minimum of 24 inches by 24 inches in cross section and supported by drilled caissons 24 inches in diameter which are placed at a minimum spacing of 6 feet on center and supporting all structural columns. The design of the grade beam and caissons should be in accordance with the recommendations of the project structural engineer, and utilize the following geotechnical parameters:

Foundations Design Criteria - Drilled Piers

The proposed additional construction, including the proposed swimming pool, may be supported in whole by drilled, cast-in-place, concrete piers which penetrate existing fill and colluvium, and are embedded into the underlying bedrock material. We anticipate that the wall loads of 1.5 kips/foot, and column loads of 10 to 50 kips will be utilized.

The drilled pier foundation for the building should gain vertical support from friction and end bearing in the native dense formational soils underlying the site. Drilled piers for residential foundations are intended to resist vertical and lateral loads due to imposed structural loads and not provide lateral stability/stabilization of slopes. The drilled piers should be at least 12 inches in diameter and should extend at least 7 feet into formational material. Drilled piers should be spaced a minimum of 3 pier diameters apart (center to center). The effects of pier groups should be evaluated when the preliminary foundation drawings are made available. Soil parameters to be used in pier and grade beam design



1. Minimum pier lengths should be 5 feet in to unweathered Ardath shale.
2. Capacities are allowable capacities (based on Factor of Safety = 2) and may be increased by one-third for short-term wind or seismic loads.
3. For uplift, use 75 percent of these capacities for single piers and 50 percent for piers in clusters.

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PIER CAPACITY CHART

Figure 4

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are provided below. All the parameters provided are computed based on soil strength only, structural strength of the piers should be checked by the structural engineer or civil engineer specializing in structural analysis.

Creep Zone: 5-foot vertical zone below a given point on the slope face, and projected upward, parallel to the slope face.

Creep Load: The creep load projected on the area of the grade beam should be taken as an equivalent fluid approach, having a density of 60 pcf. For the caisson, it should be taken as a uniform 900 pounds per linear foot of drilled pier's depth, located above the creep zone.

Point of Fixity: Located a distance of 1.5 times the caisson's diameter, below grade, or the creep zone, whichever controls.

Passive Resistance: Passive earth pressure of 300 psf per foot of depth, to a maximum value of 3,000 psf may be used to determine drilled pier depth and spacing, provided that they meet or exceed the minimum requirements stated above. To determine the total lateral resistance, the contribution of the creep prone zone above the point of fixity, to passive resistance, should be disregarded. No contribution from soil/concrete friction on the bottom of slabs should be included in passive calculations.

The upper 12 inches of passive resistance for the drilled piers should be neglected unless confined by slabs or pavement. Additional lateral resistance may be obtained from lateral pile deflection. For a ¼ inch lateral pile deflection, a lateral load of 10 percent of vertical capacity can be utilized. A more refined lateral load capacity may be provided when the pier head conditions (fixed, free), layout and elevations are provided by the structural consultant and/or architect on this project.

Allowable Axial Capacity:

Shaft capacity : 400 psf applied over the surface area of the shaft within bedrock only.

Tip capacity: 6,000 psf. Assumes clean dense tip condition.

Pier Construction

Pier holes should be drilled straight and plumb. Locations (both plan and elevation) and plumbness should be the contractors responsibility. All loose materials should be removed from the bottom of each pier hole. Concrete and steel reinforcement should be placed in each pier hole on the same day that the hole is drilled. If a caving sand condition occurs, during or after drilling, the pier hole should be cased. The bottom of the casing should be at least 4 feet below the top of the concrete as the concrete is poured and the casing is withdrawn. Dewatering would be required for concrete placement if seepage or groundwater is encountered during construction. Alternately, tremie concrete placement should be considered.

The tops of the drilled piers should be interconnected with grade beams which will aid in resisting differential foundation movement and lateral drift. In general the minimum grade beam size should be 18 inches in width and 12 inches below the finished soil subgrade. The actual design of the grade beams and reinforcement should be performed by the Structural Engineer or civil engineer specializing in structural analysis.

Based on the allowable foundation pressures recommended above, and assuming uniformity of the bedrock surface slope and consistent composition of the bedrock, we estimate that the total foundation settlement will be less than ½ inch and the differential settlement will be less than ¼ inch between adjacent piers.

Prior to construction, we should review the construction procedure proposed by the contractor. Pier excavations should be observed and approved by us prior to concrete and steel placement. Observations during pier excavations will allow us to correlate the subsurface conditions exposed during construction with that obtained from our borings and make necessary changes in the foundation support and other geotechnical design criteria, if necessary.

Drilled pier steel reinforcement cages should have spacers to allow for a minimum spacing of steel from the side of the pier excavation. During pier placement, concrete should not be allowed to free fall more than 5 feet. Concrete used in the foundation should be tested by a qualified materials testing consultant for proper slump strength and mix design.

All footing trench excavations and/or pier excavations should be observed by a representative of this office prior to placing reinforcement. Footing trench or pier soil and any excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent if not removed from the site.

Drilled Pier and Grade Beam Foundation Settlement

Drilled pier and grade beam foundations should be designed to accommodate ½ inch over a 40-foot horizontal span.

Corrosion and Concrete Mix

Testing performed in preparation of this report indicate relatively high concentrations of soil sulfate onsite. Per the UBC (ICBO, 1997) Type V cement is recommended for these soil conditions. Upon completion of grading, laboratory testing should be performed of site materials for corrosion to concrete and corrosion to steel. Additional comments may be obtained from a qualified corrosion engineer at that time.

Structural Slabs

Prior to construction of slab-on-grade floors, the upper 12 inches of the subgrade should be scarified moisture-conditioned to a near-optimum water content and compacted to at least 90 percent relative compaction and moisture conditioned as previously discussed. The slab subgrade should be nonyielding and rolled smooth prior to the placement of forms or reinforcing steel. Structural floor slabs to span unsupported between grade beams (areas of existing fill) should be at least 6 inches in thickness and reinforced at a minimum of No. 4 bars at 12 inches on center. This assumes that settlement has removed soil slab support between grade beams. Therefore, the frictional contribution of slabs to passive resistance should be neglected.

If it is necessary to protect the building from dampness caused by vapor transmission through the soil and concrete, a vapor retarder should be provided beneath the slab. A typical moisture-prevention retarder includes a capillary moisture break consisting of at least 3 inches of pea gravel overlain by a moisture-proof membrane at least 15 mils thick. To protect the moisture-proof membrane during construction, a layer of sand (2 inches thick) should be placed over the membrane. This implies a total sand layer thickness of at least 5 inches.

Additional Soil Moisture Considerations

Foundation systems and slabs "shall not allow water or water vapor to enter into the structure so as to cause damage to another building component or to limit the installation of the type of flooring materials typically used for the particular application" (State of California, 2003). Therefore, the following should be considered by the structural engineer/foundation/slab designer to mitigate the transmission of water or water vapor through the slab:

1. Concrete slab underlayment should consist of 2 inches of sand (S.E. $\geq$30), underlain by a 10-mil vapor retarder (visqueen or equivalent), with all laps sealed per the UBC/CBC (ICBO, 1997 and 2001), which is, in turn, underlain by 3 inches of pea gravel placed upon a suitable slab subgrade.

2. Concrete should have a maximum water/cement ratio of 0.50. It should be noted that the recommendation for Type V cement will likely exceed this recommendation by default.

3. Slabs should be additionally sealed with a suitable slab sealant.

4. Additional recommendations regarding water or vapor transmission should be provided by the structural engineer/slab or foundation designer.

Should these recommendations not be implemented, then full disclosure of the potential for water or vapor to pass through the foundations and slabs and resultant distress should be provided to each owner, in writing.

WALL DESIGN PARAMETERS

Conventional Retaining Walls

The design parameters provided below assume that either non expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite materials (up to and including an E.I. of 70) are used to backfill any retaining walls. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the plans. Building walls, below grade, should be water-proofed or damp-proofed, depending on the degree of moisture protection desired. The foundation system for the proposed retaining walls should be designed in accordance with the recommendations presented in this and preceding sections of this report, as appropriate (i.e., embedded into suitable bedrock). Footings should be embedded a minimum of 18 inches below adjacent grade (excluding landscape layer, 6 inches) and should be 24 inches in width. There should be no increase in bearing for footing width. Recommendations for specialty walls (i.e., crib, earthstone, geogrid, etc.) can be provided upon request, and would be based on site specific conditions.

Restrained Walls

Any retaining walls that will be restrained prior to placing and compacting backfill material or that have re-entrant or male corners, should be designed for an at-rest equivalent fluid pressure (EFP) of 65 pcf, plus any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Design parameters for walls less than 3 feet in height may be superceded by City and/or County standard design. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions due to traffic, structures, seismic events or adverse geologic conditions. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request.

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)
Level*	35	45
2 to 1	50	60
* Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall.		

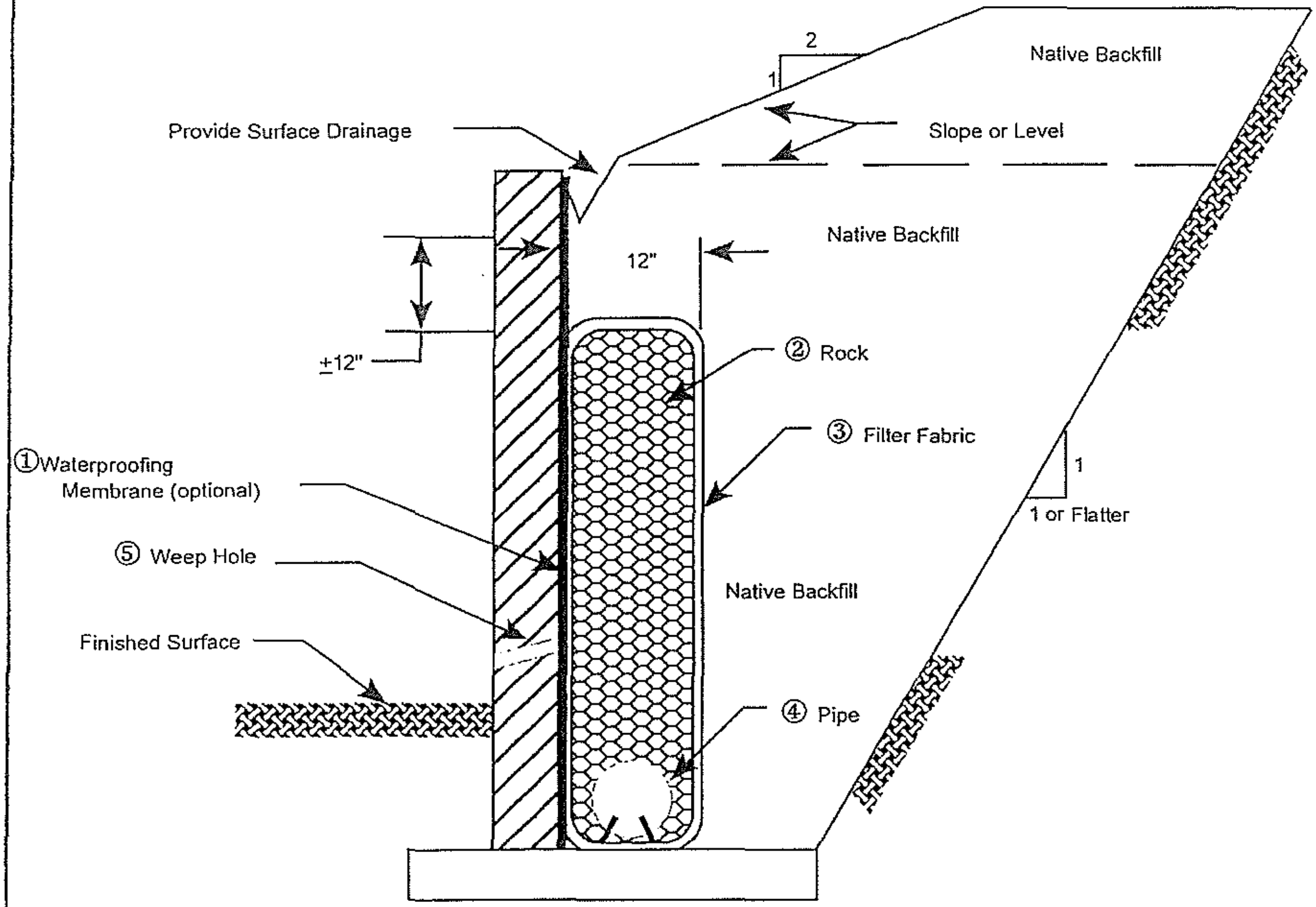
Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the back drainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140 or equivalent). For low expansive backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. For native backfill that has up to medium expansion potential, continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). Materials with an E.I. potential of greater than 65 should not be used as backfill for retaining walls. For more onerous expansive situations, backfill and drainage behind the retaining wall should conform with Detail 3 (Retaining Wall And Subdrain Detail Clean Sand Backfill).

Outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than ±100 feet apart, with a minimum of two outlets, one on each end. The use of weep holes, only, in walls higher than 2 feet, is not recommended. The surface of the backfill should be sealed by pavement or the top 18 inches compacted with native soil (E.I. $\leq$90). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints.

DETAILS

N.T.S.



① **WATERPROOFING MEMBRANE (optional):**
Liquid boot or approved equivalent.

② **ROCK:**
3/4 to 1-1/2" (inches) rock.

③ **FILTER FABRIC:**
Mirafi 140N or approved equivalent; place fabric flap behind core.

④ **PIPE:**
4" (inches) diameter perforated PVC. schedule 40 or approved alternative with minimum of 1% gradient to proper outlet point (Perforations down).

⑤ **WEEP HOLE:**
Minimum 2" (inches) diameter placed at 20' (feet) on centers along the wall, and 3" (inches) above finished surface (No weep holes for basement walls.).

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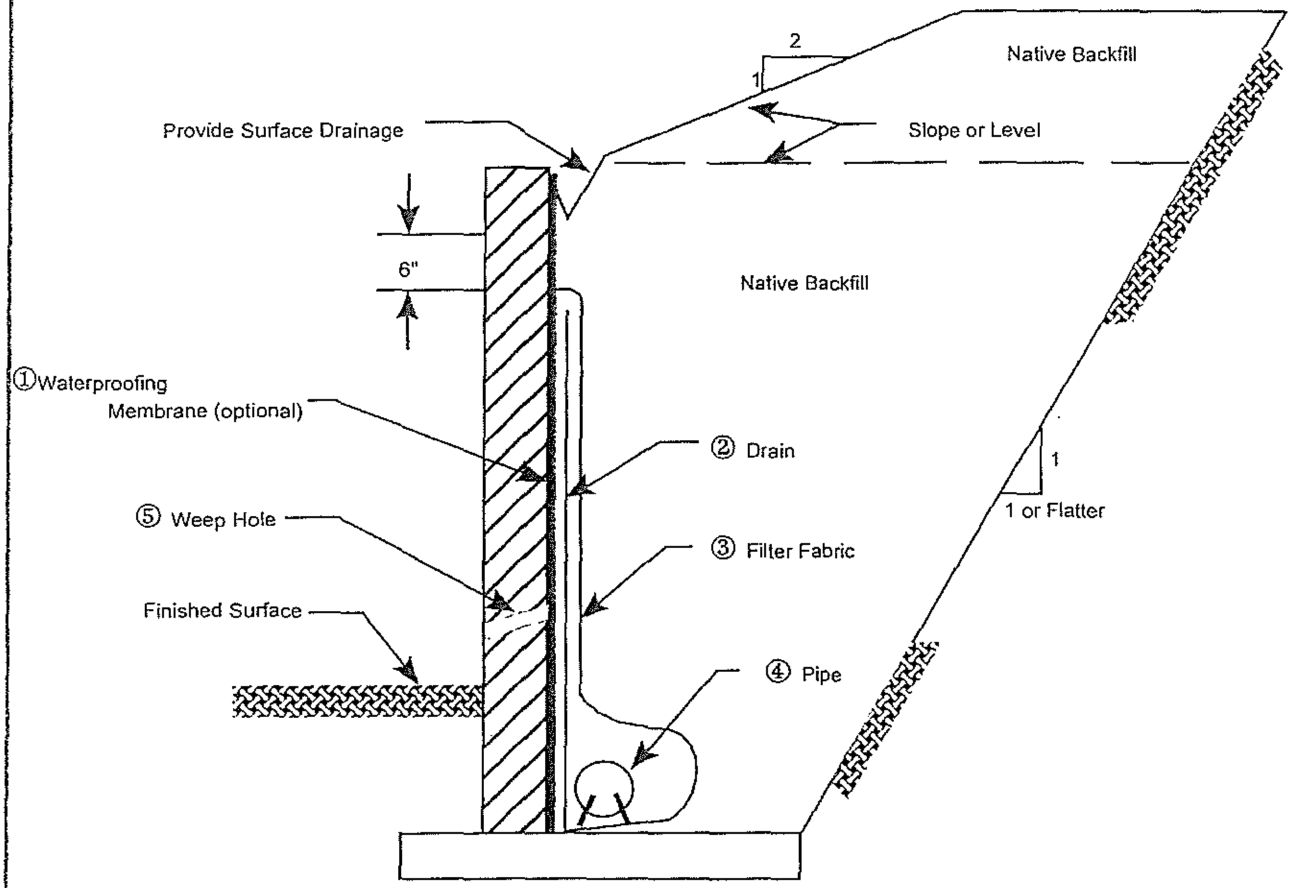
TYPICAL RETAINING WALL BACKFILL AND DRAINAGE DETAIL

DETAIL 1

Geotechnical • Coastal • Geologic • Environmental

DETAILS

N . T . S .



① **WATERPROOFING MEMBRANE (optional):**
Liquid boot or approved equivalent.

② **DRAIN:**
Miradrain 6000 or J-drain 200 or equivalent for non-waterproofed walls.
Miradrain 6200 or J-drain 200 or equivalent for waterproofed walls (All Perforations down).

③ **FILTER FABRIC:**
Mirafi 140N or approved equivalent; place fabric flap behind core.

④ **PIPE:**
4" (inches) diameter perforated PVC. schedule 40 or approved alternative with minimum of 1% gradient to proper outlet point.

⑤ **WEEP HOLE:**
Minimum 2" (inches) diameter placed at 20' (feet) on centers along the wall, and 3" (inches) above finished surface. (No weep holes for basement walls.)

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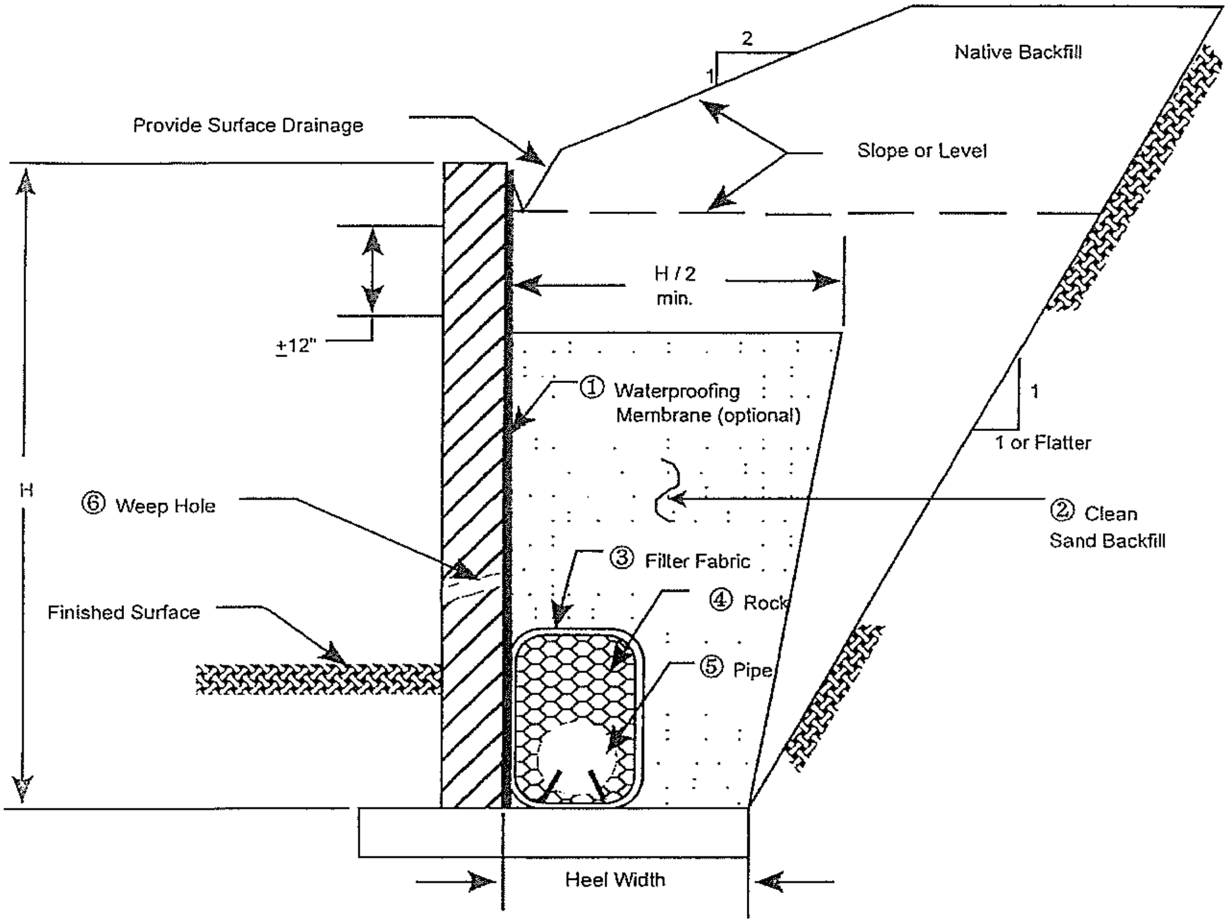
RETAINING WALL BACKFILL
AND SUBDRAIN DETAIL
GEOTEXTILE DRAIN

DETAIL 2

Geotechnical • Coastal • Geologic • Environmental

DETAILS

N . T . S .



① **WATERPROOFING MEMBRANE (optional):**
Liquid boot or approved equivalent.

② **CLEAN SAND BACKFILL:**
Must have sand equivalent value of 30 or greater; can be densified by water jetting.

③ **FILTER FABRIC:**
Mirafi 140N or approved equivalent.

④ **ROCK:**
1 cubic foot per linear feet of pipe or 3/4 to 1-1/2" (inches) rock.

⑤ **PIPE:**
4" (inches) diameter perforated PVC. schedule 40 or approved alternative with minimum of 1% gradient to proper outlet point (Perforations down).

⑥ **WEEP HOLE:**
Minimum 2" (inches) diameter placed at 20' (feet) on centers along the wall, and 3" (inches) above finished surface. (No weep holes for basement walls.)

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RETAINING WALL AND SUBDRAIN DETAIL
CLEAN SAND BACKFILL

DETAIL 3

Geotechnical • Coastal • Geologic • Environmental

Wall/Retaining Wall Footing Transitions

Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Should wall footings transition from cut to fill, the civil designer may specify either:

a) A minimum of a 2-foot overexcavation and recompaction of cut materials for a distance of 2H, from the point of transition.

b) Increase of the amount of reinforcing steel and wall detailing (i.e., expansion joints or crack control joints) such that a angular distortion of 1/360 for a distance of 2H on either side of the transition may be accommodated. Expansion joints should be placed no greater than 20 feet on-center, in accordance with the structural engineer's/wall designer's recommendations, regardless of whether or not transition conditions exist. Expansion joints should be sealed with a flexible, non-shrink grout.

c) Embed the footings entirely into native formational material (i.e., deepened footings).

If transitions from cut to fill transect the wall footing alignment at an angle of less than 45 degrees (plan view), then the designer should follow recommendation "a" (above) and until such transition is between 45 and 90 degrees to the wall alignment.

POOL RECOMMENDATIONS

As indicated in the previous sections of this report, GSI has proposed two alternative for treatment of the existing undocumented fill. Once Alternative A or B has been elected, the following pool design recommendations may be followed.

1. Due to the presence of expansive soils at the subject site, the equivalent fluid pressure to be used for the pool design should be 125 pcf for pool walls with level backfill. No sloped backfill condition is recommended within "D" from the edge of the pool, where "D" is the depth of the pool and shell. In addition, backdrains should be provided behind pool walls subjacent to any slopes. A sump may be necessary if backdrains cannot flow to an approved outlet via gravity. Sumps should be designed by the project civil engineer or architect and not allow the localized saturation of soils. This pressure assumes that expansive soils are not located any closer than 3 feet behind/below the pool shell.

2. Passive earth pressure may be computed as an equivalent fluid having a density of 200 pcf to a maximum lateral earth pressure of 2,000 psf.

3. An allowable coefficient of friction between soil and concrete of 0.30 may be used with the dead load forces.

4. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third. No friction component should be used with drilled pier caps.

5. Where pools are planned near structures (footings, landscape features, etc.), appropriate surcharge loads need to be incorporated into design and construction by the project design civil engineer.

6. The entire pool wall should be designed as "free standing" and be capable of supporting the water in the pool without soil support. The pool bottom (estimated to be a maximum of 8 feet below current grade plus the shell thickness) should be designed to withstand a potential slab movement of up to 1 to 2 inches, due to the expansive characteristics of the underlying bedrock and the anticipated varying depth of the pool.

7. The soil beneath the pool/spa bottom should consist of a 3-foot thick layer of very low to low expansive soils (E.I. = 0 to 50) that have been compacted to a minimum relative compaction of 90 percent to minimize damage due to expansive soil forces. If a fill/bedrock transition occurs beneath the pool bottom, as is likely, the bedrock should be overexcavated to a minimum depth of 3 feet, and be replaced as very low to low expansive (E.I. = 0 to 50) compacted fill. The lateral extent of the overexcavation should be 5 feet, minimally. Prior to placing fill within the overexcavation for the pool, a durable, non-permeable membrane should be placed on the overexcavation bottom.

8. Hydrostatic pressure relief valves should be incorporated into the pool and spa designs.

9. All fittings and pipe joints, particularly fittings in the side of the pool or spa, should be properly sealed to prevent water from leaking into the adjacent soils materials. Trenches and trench bottoms for pool/water feature improvements should be sloped to drain (in the event of a leak) away from the pool and existing and planned structures.

10. An elastic expansion joint (waterproof sealant) should be installed to prevent water from seeping into the soil at all deck joints.

11. A reinforced grade beam should be placed around the skimmer to provide support and mitigate cracking around the skimmer face.

12. In order to reduce unsightly cracking, deck slabs should be minimally reinforced with No. 3 reinforcing bars at 18 inches on center. All slab reinforcement should be supported to ensure proper mid-slab positioning during the placement of concrete.

13. Pool bottom or deck slabs should be founded entirely on a 3-foot thick layer of properly compacted very low to low expansive fill (E.I. = 0 to 50). Fill/bedrock transitions should be mitigated as specified above. Fill should be compacted to achieve a minimum 90 percent relative compaction. Prior to placement of concrete slabs, subgrade soils below the pool decking should be throughly watered to achieve a moisture content that is at least 10 percent above (1.1 times for very low to low expansive solis) the soil's optimum moisture content, to a depth of at least 3 feet below the bottom of slabs. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and minimize the development of unsightly shrinkage cracks.

14. In order to reduce unsightly cracking, the outer edges of pool decking to be bordered by landscaping, and the edges immediately adjacent to the pool/spa, should be underlain by 8-inch wide concrete cutoff walls (thickened edge) extending to a depth of at least 12 inches below the bottom of the slabs to prevent excessive infiltration of water under the pool deck. These thickened edges should be reinforced with two No. 4 bars, one at the top and one at the bottom. Deck slabs may be minimally reinforced with No. 3 reinforcing bars placed at 18 inches on center, in both directions. All slab reinforcement should be supported to ensure proper mid-slab positioning during the placement of concrete.

15. Surface and shrinkage cracking of the finish slab may be reduced or eliminated if a low slump and water-cement ratio are maintained during concrete placement. Excessive water added to concrete prior to placement is likely to cause shrinkage cracking.

16. Joint and sawcut locations for the pool deck should be determined by the design engineer and/or contractor. However, spacings should not exceed 10 feet.

17. It is imperative that adequate provisions for surface drainage are incorporated by the developer into their overall improvement scheme. Ponding water, ground saturation, and flows over slope faces are all conditions which must be avoided.

18. Soil generated from the pool, spa, and trench excavations to be used onsite should be compacted regardless of where it is to be placed. This material must not alter positive drainage patterns away from structural areas and toward the street or approved drainage devices.

19. Local irrigation and drainage should be diverted from all flatwork areas. Area drains and swales should be utilized to reduce the amount of subsurface water intrusion beneath the flatwork areas.

20. Bottoms of all pool utility trenches should be sloped away from the pool. Pump utility intersections should be evaluated by a mechanical consultant in light of the expansive soil conditions and the consequences of utility leakage.

21. All pipe fittings/drains (pressure and gravity pipes), gas lines, electric lines to/from the pool/ponds/fountains should be designed for up to 8 inches of vertical or lateral deformation on a cyclic basis.

22. Free standing support posts (flags, slides, light poles, etc.) should be minimally supported on circular footings embedded 3 feet into select earth material and have a minimum diameter of 1 foot. The design skin friction should be 150 psf.

DRIVEWAY, FLATWORK, AND OTHER IMPROVEMENTS

The soil materials on site may be expansive. The effects of expansive soils are cumulative, and typically occur over the lifetime of any improvements. On relatively level areas, when the soils are allowed to dry, the dessication and swelling process tends to cause heaving and distress to flatwork and other improvements. The resulting potential for distress to improvements may be reduced, but not totally eliminated. To that end, it is recommended that the developer should notify any homeowners and/or any other interested/affected parties of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. The subgrade area for concrete slabs should be compacted to achieve a minimum 90 percent relative compaction, and then be presoaked to 2 to 3 percentage points above (or 125 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. If very low expansive soils are present, only optimum moisture content, or greater, is required and specific presoaking is not warranted. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.

2. Concrete slabs should be cast over a non-yielding surface, consisting of a 4-inch layer of crushed rock, gravel, or clean sand, that should be compacted and level prior to pouring concrete. If very low expansive soils are present, the rock or gravel or sand may be deleted. The layer or subgrade should be wet-down completely prior to pouring concrete, to minimize loss of concrete moisture to the surrounding earth materials.

3. Exterior slabs should be a minimum of 4 inches thick. Driveway slabs and approaches should additionally have a thickened edge (12 inches) adjacent to all landscape areas, to help impede infiltration of landscape water under the slab.

4. The use of transverse and longitudinal control joints are recommended to help control slab cracking due to concrete shrinkage or expansion. Two ways to mitigate such cracking are: a) add a sufficient amount of reinforcing steel, increasing tensile strength of the slab; and, b) provide an adequate amount of control and/or expansion joints to accommodate anticipated concrete shrinkage and expansion.

   In order to reduce the potential for unsightly cracks, slabs should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. If subgrade soils within the top 7 feet from finish grade are very low expansive soils (i.e., E.I. ≤20), then 6x6-W1.4xW1.4 welded-wire mesh may be substituted for the rebar, provided the reinforcement is placed on chairs, at slab mid-height. The exterior slabs should be scored or saw cut, ½ to ⅜ inches deep, often enough so that no section is greater than 10 feet by 10 feet. For sidewalks or narrow slabs, control joints should be provided at intervals of every 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

5. No traffic should be allowed upon the newly poured concrete slabs until they have been properly cured to within 75 percent of design strength. Concrete compression strength should be a minimum of 2,500 psi.

6. Driveways, sidewalks, and patio slabs adjacent to the house should be separated from the house with thick expansion joint filler material. In areas directly adjacent to a continuous source of moisture (i.e., irrigation, planters, etc.), all joints should be additionally sealed with flexible mastic.

7. Planters and walls should not be tied to the house.

8. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied in at least two directions. If very low expansion soils are present, footings need only be tied in one direction.

9. Any masonry landscape walls that are to be constructed throughout the property should be grouted and articulated in segments no more than 20 feet long. These segments should be keyed or doweled together.

10. Utilities should be enclosed within a closed utilidor (vault) or designed with flexible connections to accommodate differential settlement and expansive soil conditions.

11. Positive site drainage should be maintained at all times. Finish grade on the lots should provide a minimum of 1 to 2 percent fall to the street, as indicated herein. It should be kept in mind that drainage reversals could occur, including post-construction settlement, if relatively flat yard drainage gradients are not periodically maintained by the homeowner and/or other interested/affected parties.

12. Air conditioning (A/C) units should be supported by slabs that are incorporated into the building foundation or constructed on a rigid slab with flexible couplings for plumbing and electrical lines. A/C waste water lines should be drained to a suitable non-erosive outlet.

13. Shrinkage cracks could become excessive if proper finishing and curing practices are not followed. Finishing and curing practices should be performed per the Portland Cement Association Guidelines. Mix design should incorporate rate of curing for climate and time of year, sulfate content of soils, corrosion potential of soils, and fertilizers used on site.

DEVELOPMENT CRITERIA

Slope Deformation

Compacted fill slopes designed using customary factors of safety for gross or surficial stability and constructed in general accordance with the design specifications should be expected to undergo some differential vertical heave or settlement in combination with differential lateral movement in the out-of-slope direction, after grading. This post-construction movement occurs in two forms: slope creep, and lateral fill extension (LFE). Slope creep is caused by alternate wetting and drying of the fill soils which results in slow downslope movement. This type of movement is expected to occur throughout the life of the slope, and is anticipated to potentially affect improvements or structures (e.g., separations and/or cracking), placed near the top-of-slope, up to a maximum distance of approximately 15 feet from the top-of-slope, depending on the slope height. This movement generally results in rotation and differential settlement of improvements located within the creep zone. LFE occurs due to deep wetting from irrigation and rainfall on slopes comprised of expansive materials. Although some movement should be expected, long-term movement from this source may be minimized, but not eliminated, by placing the fill throughout the slope region, wet of the fill's optimum moisture content.

It is generally not practical to attempt to eliminate the effects of either slope creep or LFE. Suitable mitigative measures to reduce the potential of lateral deformation typically include: setback of improvements from the slope faces (per the 1997 UBC and/or adopted California Building Code), positive structural separations (i.e., joints) between improvements, and stiffening and deepening of foundations. Expansion joints in walls should be placed no greater than 20 feet on-center, and in accordance with the structural engineer's recommendations. All of these measures are recommended for design of structures and improvements. The ramifications of the above conditions, and recommendations for mitigation, should be provided to each homeowner and/or any other interested/affected parties.

Slope Maintenance and Planting

Water has been shown to weaken the inherent strength of all earth materials. Slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Over-watering should be avoided as it adversely affects site improvements, and causes perched groundwater conditions. Graded slopes constructed utilizing onsite materials would be erosive. Eroded debris may be minimized and surficial slope stability enhanced by establishing and maintaining a suitable vegetation cover soon after construction. Compaction to the face of fill slopes would tend to minimize short-term erosion until vegetation is established. Plants selected for landscaping should be light weight, deep rooted types that require little water and are capable of surviving the prevailing climate. Jute-type matting or other fibrous covers may aid in allowing the establishment of a sparse plant cover. Utilizing plants other than those recommended above will increase the potential for perched water, staining, mold, etc., to develop. A rodent control program to prevent burrowing should be implemented. Irrigation of natural (ungraded) slope areas is generally not recommended. These recommendations regarding plant type, irrigation practices, and rodent control should be provided to each homeowner and/or other interested/affected parties. Over-steepening of slopes should be avoided during building construction activities and landscaping.

Drainage

Adequate lot surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on a lot, and especially near structures and tops of slopes. Lot surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage within lots and common areas should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond and/or seep into the ground. In general, the area within 5 feet around a structure should slope away from the structure. We recommend that unpaved lawn and landscape areas have a minimum gradient of 1 percent sloping away from structures, and whenever possible, should be above adjacent paved areas. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, pools, spas, etc.). Pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, down spouts, or other appropriate means may be utilized to control roof drainage. Down spouts, or drainage devices should outlet a minimum of 5 feet from structures or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for minimizing this effect could be provided upon request.

Erosion Control

Cut and fill slopes will be subject to surficial erosion during and after grading. Onsite earth materials have a moderate to high erosion potential. Consideration should be given to providing hay bales and silt fences for the temporary control of surface water, from a geotechnical viewpoint.

Landscape Maintenance

Only the amount of irrigation necessary to sustain plant life should be provided. Over-watering the landscape areas will adversely affect proposed site improvements. We would recommend that any proposed open-bottom planters adjacent to proposed structures be eliminated for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be utilized. An outlet placed in the bottom of the planter, could be installed to direct drainage away from structures or any exterior concrete flatwork. If planters are constructed adjacent to structures, the sides and bottom of the planter should be provided with a moisture retarder to prevent penetration of irrigation water into the subgrade. Provisions should be made to drain the excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters. Graded slope areas should be planted with drought resistant vegetation. Consideration should be given to the type of vegetation chosen and their potential effect upon surface improvements (i.e., some trees will have an effect on concrete flatwork with their extensive root systems). From a geotechnical standpoint leaching is not recommended for establishing landscaping. If the surface soils are processed for the purpose of adding amendments, they should be recompacted to 90 percent minimum relative compaction.

Gutters and Downspouts

As previously discussed in the drainage section, the installation of gutters and downspouts should be considered to collect roof water that may otherwise infiltrate the soils adjacent to the structures. If utilized, the downspouts should be drained into PVC collector pipes or other non-erosive devices (e.g., paved swales or ditches; below grade, solid tight-lined PVC pipes; etc.), that will carry the water away from the house, to an appropriate outlet, in accordance with the recommendations of the design civil engineer. Downspouts and gutters are not a requirement; however, from a geotechnical viewpoint, provided that positive drainage is incorporated into project design (as discussed previously).

Subsurface and Surface Water

Subsurface and surface water are not anticipated to affect site development, provided that the recommendations contained in this report are incorporated into final design and construction and that prudent surface and subsurface drainage practices are incorporated into the construction plans. Perched groundwater conditions along zones of contrasting permeabilities may not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities, and should be anticipated. Should perched groundwater conditions develop, this office could assess the affected area(s) and provide the appropriate recommendations to mitigate the observed groundwater conditions. Groundwater conditions may change with the introduction of irrigation, rainfall, or other factors.

Site Improvements

If in the future, any additional improvements (e.g., pools, spas, etc.) are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. Pools and/or spas should not be constructed without specific design and construction recommendations from GSI, and this construction recommendation should be provided to the homeowners and/or other interested/affected parties. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, utility trench and retaining wall backfills, flatwork, etc.

Tile Flooring

Tile flooring can crack, reflecting cracks in the concrete slab below the tile, although small cracks in a conventional slab may not be significant. Therefore, the designer should consider additional steel reinforcement for concrete slabs-on-grade where tile will be placed. The tile installer should consider installation methods that reduce possible cracking of the tile such as slipsheets. Slipsheets or a vinyl crack isolation membrane (approved by the Tile Council of America/Ceramic Tile Institute) are recommended between tile and concrete slabs on grade.

Additional Grading

This office should be notified in advance of any fill placement, supplemental regrading of the site, or trench backfilling after rough grading has been completed. This includes completion of grading in the street, driveway approaches, driveways, parking areas, and utility trench and retaining wall backfills.

Footing Trench Excavation

All footing excavations should be observed by a representative of this firm subsequent to trenching and prior to concrete form and reinforcement placement. The purpose of the observations is to evaluate that the excavations have been made into the recommended bearing material and to the minimum widths and depths recommended for construction. If loose or compressible materials are exposed within the footing excavation, a deeper footing or removal and recompaction of the subgrade materials would be recommended at that time. Footing trench spoil and any excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent, if not removed from the site.

Trenching/Temporary Construction Backcuts

Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees [except as specifically superceded within the text of this report]), should be anticipated. All excavations should be observed by an engineering geologist or soil engineer from GSI, prior to workers entering the excavation or trench, and minimally conform to CAL-OSHA, state, and local safety codes. Should adverse conditions exist, appropriate recommendations would be offered at that time. The above recommendations should be provided to any contractors and/or subcontractors, or homeowners, etc., that may perform such work.

Utility Trench Backfill

1. All interior utility trench backfill should be brought to at least 2 percent above optimum moisture content and then compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard. As an alternative for shallow (12-inch to 18-inch) under-slab trenches, sand having a sand equivalent value of 30 or greater may be utilized and jetted or flooded into place. Observation, probing and testing should be provided to evaluate the desired results.

2. Exterior trenches adjacent to, and within areas extending below a 1:1 plane projected from the outside bottom edge of the footing, and all trenches beneath hardscape features and in slopes, should be compacted to at least 90 percent of the laboratory standard. Sand backfill, unless excavated from the trench, should not be used in these backfill areas. Compaction testing and observations, along with probing, should be accomplished to evaluate the desired results.

3. All trench excavations should conform to CAL-OSHA, state, and local safety codes.

4. Utilities crossing grade beams, perimeter beams, or footings should either pass below the footing or grade beam utilizing a hardened collar or foam spacer, or pass through the footing or grade beam in accordance with the recommendations of the structural engineer.

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

We recommend that observation and/or testing be performed by GSI at each of the following construction stages:

- During grading/recertification.
- During excavation.
- During placement of subdrains, toe drains, or other subdrainage devices, prior to placing fill and/or backfill.
- After excavation of building footings, retaining wall footings, and free standing walls footings, prior to the placement of reinforcing steel or concrete.
- Prior to pouring any slabs or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor retarders (i.e., visqueen, etc.).
- During retaining wall subdrain installation, prior to backfill placement.
- During placement of backfill for area drain, interior plumbing, utility line trenches, and retaining wall backfill.
- During slope construction/repair.
- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.
- When any developer or homeowner improvements, such as flatwork, spas, pools, walls, etc., are constructed, prior to construction. GSI should review such plans prior to construction.
- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, and/or to comply with code requirements.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

The design civil engineer, structural engineer, post-tension designer, architect, landscape architect, wall designer, etc., should review the recommendations provided herein, incorporate those recommendations into all their respective plans, and by explicit reference, make this report part of their project plans. This report presents minimum design criteria for the design of slabs, foundations and other elements possibly applicable to the project. These criteria should not be considered as substitutes for actual designs by the structural engineer/designer. Please note that the recommendations contained herein are not intended to preclude the transmission of water or vapor through the slab or foundation. The structural engineer/foundation and/or slab designer should provide recommendations to not allow water or vapor to enter into the structure so as to cause damage to another building component, or so as to limit the installation of the type of flooring materials typically used for the particular application.

The structural engineer/designer should analyze actual soil-structure interaction and consider, as needed, bearing, expansive soil influence, and strength, stiffness and deflections in the various slab, foundation, and other elements in order to develop appropriate, design-specific details. As conditions dictate, it is possible that other influences will also have to be considered. The structural engineer/designer should consider all applicable codes and authoritative sources where needed. If analyses by the structural engineer/designer result in less critical details than are provided herein as minimums, the minimums presented herein should be adopted. It is considered likely that some, more restrictive details will be required.

If the structural engineer/designer has any questions or requires further assistance, they should not hesitate to call or otherwise transmit their requests to GSI. In order to mitigate potential distress, the foundation and/or improvement's designer should confirm to GSI and the governing agency, in writing, that the proposed foundations and/or improvements can tolerate the amount of differential settlement and/or expansion characteristics and other design criteria specified herein.

PLAN REVIEW

Final project plans (grading, precise grading, foundation, retaining wall, landscaping, etc.), should be reviewed by this office prior to construction, so that construction is in accordance with the conclusions and recommendations of this report. Based on our review, supplemental recommendations and/or further geotechnical studies may be warranted.

LIMITATIONS

The materials encountered on the project site and utilized for our analysis are believed representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during mass grading. Site conditions may vary due to seasonal changes or other factors.

Inasmuch as our study is based upon our review and engineering analyses and laboratory data, the conclusions and recommendations are professional opinions. These opinions have been derived in accordance with current standards of practice, and no warranty, either express or implied, is given. Standards of practice are subject to change with time. GSI assumes no responsibility or liability for work or testing performed by others, or their inaction; or work performed when GSI is not requested to be onsite, to evaluate if our recommendations have been properly implemented. Use of this report constitutes an agreement and consent by the user to all the limitations outlined above, notwithstanding any other agreements that may be in place. In addition, this report may be subject to review by the controlling authorities. Thus, this report brings to completion our scope of services for this portion of the project. All samples will be disposed of after 30 days, unless specifically requested by the Client, in writing.

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions			Group Symbols	Typical Names
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with	GM	Silty gravels gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands more than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils 50% or more passes No. 200 sieve	Silts and Clays Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, mucic, and other highly organic soils

CONSISTENCY OR RELATIVE DENSITY

CRITERIA

Standard Penetration Test

Penetration Resistance N (blows/ft)	Relative Density
0 - 4	Very loose
4 - 10	Loose
10 - 30	Medium
30 - 50	Dense
> 50	Very dense

Standard Penetration Test

Penetration Resistance N (blows/ft)	Consistency	Unconfined Compressive Strength (tons/ft^2)
<2	Very Soft	<0.25
2 - 4	Soft	0.25 - .050
4 - 8	Medium	0.50 - 1.00
8 - 15	Stiff	1.00 - 2.00
15 - 30	Very Stiff	2.00 - 4.00
>30	Hard	>4.00

3" 3/4" #4 #10 #40 #200 U.S. Standard Sieve

Unified Soil Classification	Cobbles	Gravel		Sand			Silt or Clay
		coarse	fine	coarse	medium	fine	

MOISTURE CONDITIONS

Dry	Absence of moisture: dusty, dry to the touch
Slightly Moist	Below optimum moisture content for compaction
Moist	Near optimum moisture content
Very Moist	Above optimum moisture content
Wet	Visible free water; below water table

MATERIAL QUANTITY

trace	0 - 5 %
few	5 - 10 %
little	10 - 25 %
some	25 - 45 %

OTHER SYMBOLS

C	Core Sample
S	SPT Sample
B	Bulk Sample
▽	Groundwater
Qp	Pocket Penetrometer

BASIC LOG FORMAT:
Group name, Group symbol, (grain size), color, moisture, consistency or relative density. Additional comments: odor, presence of roots, mica, gypsum, coarse grained particles, etc.

EXAMPLE:
Sand (SP), fine to medium grained, brown, moist, loose, trace silt, little fine gravel, few cobbles up to 4" in size, some hair roots and rootlets.

GeoSoils, Inc.

BORING LOG

W.O. 4971-A-SC

PROJECT: TASSVIRI
2585 Calle del Oro, La Jolla

BORING B-1 SHEET 1 OF 2

DATE EXCAVATED 2-6-06

SAMPLE METHOD: 6" Hollow Stem Auger

Approx. Elevation: 1' MSL

▩ Standard Penetration Test

▨ Undisturbed, Ring Sample

∇ Groundwater

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
0				SC				**TOPSOIL:** @ 0' CLAYEY SAND, brown, wet, loose.
1				CL				**ARDATH SHALE:** @ 1' SANDY CLAYSTONE, light brown, moist, hard.
5	▩	▨	47		109.4	13.4	70	@ 5' As per 1', some gypsum, fractured. Fracture trending NE, dipping approximately 60 degrees.
10		▨	75		112.0	18.4	98	@ 10' As per 5', wet.
15		▨	80		108.2	17.0	85	@ 15' As per 10', bedding in oriented sample observed dipping 3 degrees northeast.
20								
25	▩	▨	50		107.4	18 2	90	@ 25' As per 15', gypsum less abundant.

BORING LOG

GeoSoils, Inc.

W.O. 4971-A-SC

PROJECT: TASSVIRI
2585 Calle del Oro, La Jolla

BORING B-1 SHEET 2 OF 2

DATE EXCAVATED 2-6-06

SAMPLE METHOD: 6" Hollow Stem Auger

Approx. Elevation: 1' MSL

▩ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

▽ *Groundwater*

Depth (ft)	Sample		Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)		Description of Material
	Bulk	Undisturbed							
		▨	80	CL	110.9	15.7	85	▨	@ 30' As per 25', becomes light grayish brown, moist, very hard; very difficult drilling, no visible gypsum.
35		▨	75		111.2	15.9	85	▨	@ 35' As per 30'.
									Total Depth = 36' No Groundwater Encountered Backfilled 2-2-2006
40									
45									
50									
55									

BORING LOG

GeoSoils, Inc.

W.O. 4971-A-SC

PROJECT: TASSVIRI
2585 Calle del Oro, La Jolla

BORING B-2 SHEET 1 OF 1

DATE EXCAVATED 2-6-06

SAMPLE METHOD: Hand Auger/Test Pit

Approx. Elevation: ____' MSL

▩ Standard Penetration Test

▨ Undisturbed, Ring Sample

∇ Groundwater

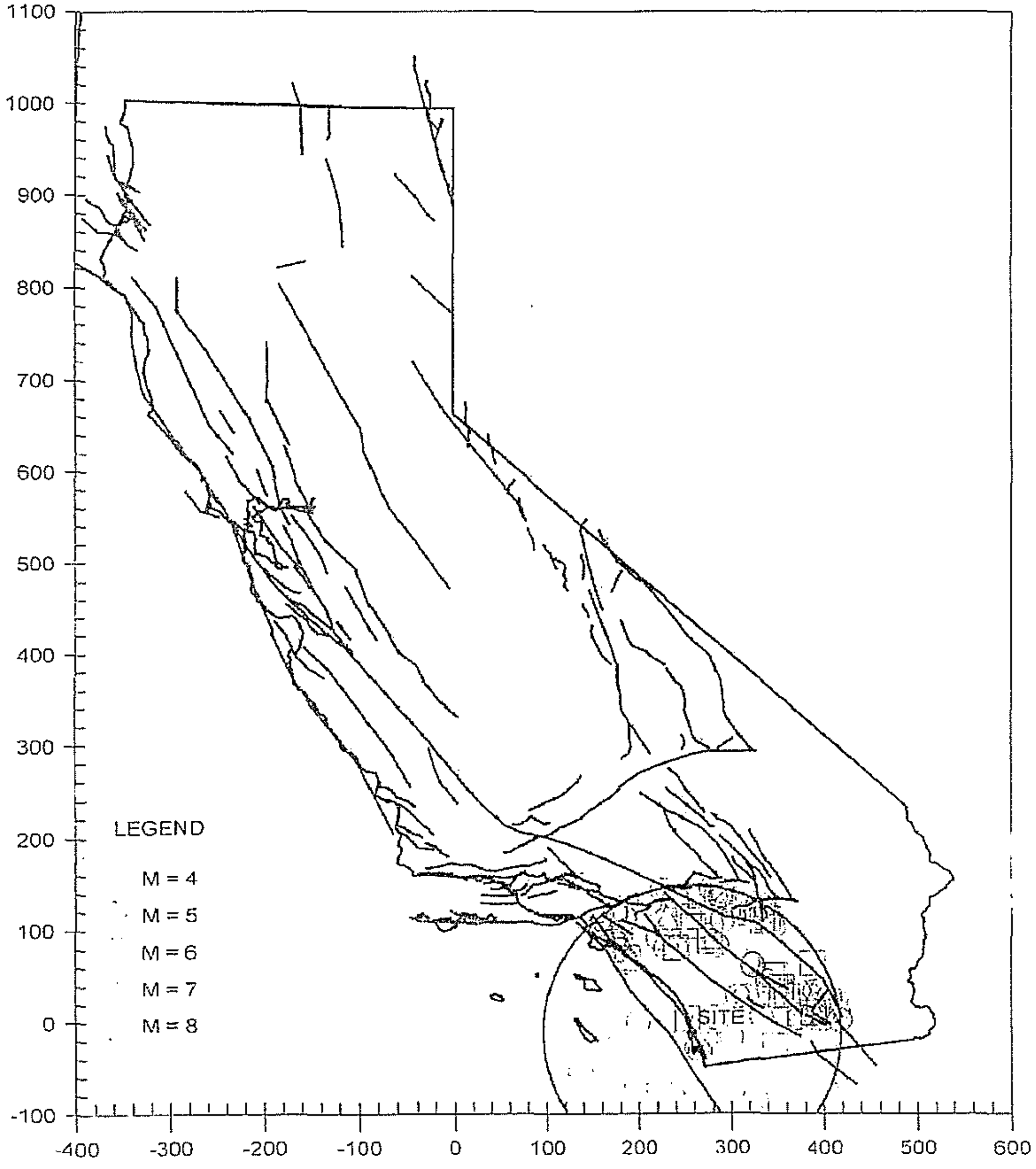
Depth (ft)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)		Description of Material
	Bulk	Undisturbed	Blows/ft.						
				SC				▨	**COLLUVIUM:** @ 0' CLAYEY SAND, brown, dry, loose.
				CL				▨	**ARDATH SHALE:** @ 3' CLAYSTONE, brown, moist, stiff; fractured, dipping approximately 55 degress NE.
5									
									Total Depth = 7' No Groundwater Encountered Backfilled 2-6-06
10									
15									
20									
25									

APPENDIX C

EQFAULT, EQSEARCH, AND FRISKSP

EARTHQUAKE EPICENTER MAP

Tassviri



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                 *************************
                 *                       *
                 *     E Q S E A R C H   *
                 *                       *
                 *      Version 3.00     *
                 *                       *
                 *************************

                     ESTIMATION OF
                PEAK ACCELERATION FROM
              CALIFORNIA EARTHQUAKE CATALOGS


JOB NUMBER: 4971-A-SC
                                                  DATE: 12-12-2006

JOB NAME: Tassviri

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT


SITE COORDINATES:
   SITE LATITUDE:  32.8550
   SITE LONGITUDE:  117.2488

SEARCH DATES:
           START DATE:   1800
           END DATE:   2006

SEARCH RADIUS:
           100.0 mi
           160.9 km

ATTENUATION RELATION:  21) Sadigh et al. (1997) Horiz. - Rock
   UNCERTAINTY (M=Median, S=Sigma): S       Number of Sigmas:  1.0
   ASSUMED SOURCE TYPE:  DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND:   1  Depth Source:  A
   Basement Depth:  5.00 km     Campbell SSR:        Campbell SHR:
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km):  0.0
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                         EARTHQUAKE SEARCH RESULTS
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Page  1
-------------------------------------------------------------------------------
    |         |         |           |  TIME    |     |     |  SITE  |SITE|  APPROX.
FILE|  LAT.   |  LONG.  |   DATE    |  (UTC)   |DEPTH|QUAKE|  ACC.  | MM |  DISTANCE
CODE| NORTH   |  WEST   |           | H M Sec  | (km)| MAG.|   g    |INT.|  mi   [km]
----+---------+---------+-----------+----------+-----+-----+--------+----+-------------
MGI |32.8000|117.1000|05/25/1803|  0 0  0.0|  0.0| 5.00| 0.175  |VIII|  9.4( 15.2)
DMG |33.0000|117.3000|11/22/1800|2130  0.0|  0.0| 6.50| 0.387  |  X | 10.4( 16.8)
DMG |32.7000|117.2000|05/27/1862|20 0  0.0|  0.0| 5.90| 0.254  | IX | 11.1( 17.8)
T-A |32.6700|117.1700|10/21/1862|  0 0  0.0| · 0.0| 5.00| 0.112  | VII| 13.6( 21.8)
T-A |32.6700|117.1700|12/00/1856|  0 0  0.0|  0.0| 5.00| 0.112  | VII| 13.6( 21.8)
T-A |32.6700|117.1700|05/24/1865|  0 0  0.0|  0.0| 5.00| 0.112  | VII| 13.6( 21.8)
MGI |33.0000|117.0000|09/21/1856|  730  0.0|  0.0| 5.00| 0.078  | VII| 17.5( 28.2)
DMG |32.8000|116.8000|10/23/1894|23 3  0.0|  0.0| 5.70| 0.070  | VI | 26.3( 42.3)
PAS |32.9710|117.8700|07/13/1986|1347  8.2|  6.0| 5.30| 0.030  |  V | 36.9( 59.4)
DMG |33.2000|116.7000|01/01/1920|  235  0.0|  0.0| 5.00| 0.021  | IV | 39.7( 63.9)
T-A |32.2500|117.5000|01/13/1877|20 0  0.0|  0.0| 5.00| 0.017  | IV | 44.3( 71.2)
MGI |33.2000|116.6000|10/12/1920|1748  0.0|  0.0| 5.30| 0.022  | IV | 44.5( 71.6)
DMG |33.0000|116.4330|06/04/1940|1035  8.3|  0.0| 5.10| 0.016  | IV | 48.3( 77.8)
GSP |32.3290|117.9170|06/15/2004|222848.2| 10.0| 5.30| 0.016  | IV | 53.2( 85.6)
DMG |32.7000|116.3000|02/24/1892|  720  0.0|  0.0| 6.70| 0.043  | VI | 56.1( 90.3)
DMG |33.7000|117.4000|05/15/1910|1547  0.0|  0.0| 6.00| 0.023  | IV | 59.0( 94.9)
DMG |33.7000|117.4000|04/11/1910|  757  0.0|  0.0| 5.00| 0.010  | III| 59.0( 94.9)
DMG |33.7000|117.4000|05/13/1910|  620  0.0|  0.0| 5.00| 0.010  | III| 59.0( 94.9)
DMG |33.6990|117.5110|05/31/1938|  83455.4| 10.0| 5.50| 0.015  | IV | 60.2( 96.9)
GSP |33.5290|116.5720|06/12/2005|154146.5| 14.0| 5.20| 0.011  | III| 60.8( 97.8)
DMG |32.0000|117.5000|06/24/1939|1627  0.0|  0.0| 5.00| 0.010  | III| 60.8( 97.9)
DMG |32.0000|117.5000|05/01/1939|2353  0.0|  0.0| 5.00| 0.010  | III| 60.8( 97.9)
DMG |32.2000|116.5500|11/05/1949|  43524.0|  0.0| 5.10| 0.011  | III| 60.8( 97.9)
DMG |32.2000|116.5500|11/04/1949|204238.0|  0.0| 5.70| 0.017  | IV | 60.8( 97.9)
PAS |33.5010|116.5130|02/25/1980|104738.5| 13.6| 5.50| 0.014  | IV | 61.6( 99.2)
DMG |33.7100|116.9250|09/23/1963|144152.6| 16.5| 5.00| 0.009  | III| 61.9( 99.6)
GSP |33.5080|116.5140|10/31/2001|075616.6| 15.0| 5.10| 0.010  | III| 61.9( 99.7)
DMG |33.5000|116.5000|09/30/1916|  211  0.0|  0.0| 5.00| 0.009  | III| 62.1( 99.9)
DMG |33.3430|116.3460|04/28/1969|232042.9| 20.0| 5.80| 0.018  | IV | 62.1(100.0)
DMG |32.0830|116.6670|11/25/1934|  818  0.0|  0.0| 5.00| 0.009  | III| 63.2(101.6)
DMG |33.7500|117.0000|06/06/1918|2232  0.0|  0.0| 5.00| 0.009  | III| 63.4(102.1)
DMG |33.7500|117.0000|04/21/1918|223225.0|  0.0| 6.80| 0.037  |  V | 63.4(102.1)
DMG |32.8170|118.3500|12/26/1951|  04654.0|  0.0| 5.90| 0.018  | IV | 63.9(102.9)
DMG |33.2000|116.2000|05/28/1892|1115  0.0|  0.0| 6.30| 0.024  |  V | 65.2(104.9)
DMG |33.5750|117.9830|03/11/1933|  518  4.0|  0.0| 5.20| 0.010  | III| 65.3(105.2)
DMG |33.4000|116.3000|02/09/1890|12 6  0.0|  0.0| 6.30| 0.023  | IV | 66.5(107.1)
DMG |33.8000|117.0000|12/25/1899|1225  0.0|  0.0| 6.40| 0.025  |  V | 66.8(107.5)
DMG |33.6170|117.9670|03/11/1933|  154  7.8|  0.0| 6.30| 0.023  | IV | 67.0(107.8)
MGI |33.8000|117.6000|04/22/1918|2115  0.0|  0.0| 5.00| 0.008  | II | 68.3(109.9)
DMG |33.2830|116.1830|03/19/1954|  95429.0|  0.0| 6.20| 0.020  | IV | 68.4(110.0)
DMG |33.2830|116.1830|03/19/1954|  95556.0|  0.0| 5.00| 0.008  | II | 68.4(110.0)
DMG |33.2830|116.1830|03/23/1954|  41450.0|  0.0| 5.10| 0.008  | III| 68.4(110.0)
DMG |33.2830|116.1830|03/19/1954|102117.0|  0.0| 5.50| 0.012  | III| 68.4(110.0)
DMG |33.4080|116.2610|03/25/1937|1649  1.8| 10.0| 6.00| 0.017  | IV | 68.7(110.6)
DMG |33.6170|118.0170|03/14/1933|19 150.0|  0.0| 5.10| 0.008  | III| 68.8(110.7)
DMG |33.1900|116.1290|04/09/1968|  22859.1| 11.1| 6.40| 0.023  | IV | 68.8(110.8)
DMG |33.2170|116.1330|08/15/1945|175624.0|  0.0| 5.70| 0.013  | III| 69.2(111.4)
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                                   Tassv.OUT
DMG  |33.9000|117.2000|12/19/1880| 0 0 0.0|   0.0| 6.00| 0.016  | IV  | 72.2(116.2)
DMG  |33.1130|116.0370|04/09/1968| 3 353.5|   5.0| 5.20| 0.008  | III| 72.4(116.5)
DMG  |31.8110|117.1310|12/22/1964|205433.2|   2.3| 5.60| 0.011  | III| 72.4(116.5)
DMG  |32.9670|116.0000|10/21/1942|162654.0|   0.0| 5.00| 0.007  | II  | 72.8(117.1)
DMG  |32.9670|116.0000|10/21/1942|162213.0|   0.0| 6.50| 0.023  | IV  | 72.8(117.1)
DMG  |32.9670|116.0000|10/22/1942|181326.0|   0.0| 5.00| 0.007  | II  | 72.8(117.1)



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                         EARTHQUAKE SEARCH RESULTS
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     |       |        |          |  TIME   |     |     |  SITE  |SITE|  APPROX.
FILE|  LAT.  |  LONG.  |   DATE   |  (UTC)  |DEPTH|QUAKE|  ACC.  | MM |  DISTANCE
CODE| NORTH  |  WEST   |          | H M Sec| (km)| MAG.|   g    |INT.|  mi   [km]
- --+-------+--------+----------+--------+-----+-----+--------+----+--------------
DMG  |32.9670|116.0000|10/21/1942|162519.0|   0.0| 5.00| 0.007  | II  | 72.8(117.1)
DMG  |33.6830|118.0500|03/11/1933|  658 3.0|   0.0| 5.50| 0.010  | III| 73.5(118.3)
DMG  |32.9830|115.9830|05/23/1942|154729.0|   0.0| 5.00| 0.007  | II  | 73.9(118.9)
DMG  |33.7000|118.0670|03/11/1933|  51022.0|   0.0| 5.10| 0.007  | II  | 75.1(120.8)
DMG  |33.7000|118.0670|03/11/1933|  85457.0|   0.0| 5.10| 0.007  | II  | 75.1(120.8)
DMG  |33.2310|116.0040|05/26/1957|155933.6|  15.1| 5.00| 0.006  | II  | 76.6(123.2)
DMG  |33.7500|118.0830|03/13/1933|131828.0|   0.0| 5.30| 0.008  | II  | 78.3(126.1)
DMG  |33.7500|118.0830|03/11/1933|  230 0.0|   0.0| 5.10| 0.007  | II  | 78.3(126.1)
DMG  |33.7500|118.0830|03/11/1933|  323 0.0|   0.0| 5.00| 0.006  | II  | 78.3(126.1)
DMG  |33.7500|118.0830|03/11/1933|  2 9 0.0|   0.0| 5.00| 0.006  | II  | 78.3(126.1)
DMG  |33.7500|118.0830|03/11/1933|  910 0.0|   0.0| 5.10| 0.007  | II  | 78.3(126.1)
DMG  |31.8670|116.5710|02/27/1937|  12918.4|  10.0| 5.00| 0.006  | II  | 78.8(126.9)
DMG  |33.9500|116.8500|09/28/1946|  719 9.0|   0.0| 5.00| 0.006  | II  | 79.0(127.2)
DMG  |34.0000|117.2500|07/23/1923|  73026.0|   0.0| 6.25| 0.016  | IV  | 79.1(127.2)
DMG  |32.5000|118.5500|02/24/1948|  81510.0|   0.0| 5.30| 0.007  | II  | 79.5(127.9)
MGI  |34.0000|117.5000|12/16/1858|10 0 0.0|   0.0| 7.00| 0.029  |  V  | 80.4(129.3)
DMG  |33.7830|118.1330|10/02/1933|  91017.6|   0.0| 5.40| 0.008  | II  | 81.9(131.8)
PAS  |33.0130|115.8390|11/24/1987|131556.5|   2.4| 6.00| 0.012  | III| 82.4(132.6)
DMG  |33.0000|115.8330|01/08/1946|185418.0|   0.0| 5.40| 0.008  | II  | 82.7(133.0)
DMG  |33.9760|116.7210|06/12/1944|104534.7|  10.0| 5.10| 0.006  | II  | 83.2(133.8)
DMG  |33.0330|115.8210|09/30/1971|224611.3|   8.0| 5.10| 0.006  | II  | 83.6(134.6)
DMG  |33.1830|115.8500|04/25/1957|222412.0|   0.0| 5.10| 0.006  | II  | 84.1(135.3)
DMG  |33.9940|116.7120|06/12/1944|111636.0|  10.0| 5.30| 0.007  | II  | 84.5(136.0)
MGI  |34.1000|117.3000|07/15/1905|2041 0.0|   0.0| 5.30| 0.006  | II  | 86.0(138.4)
DMG  |33.7830|118.2500|11/14/1941|  84136.3|   0.0| 5.40| 0.007  | II  | 86.3(138.8)
PAS  |33.0820|115.7750|11/24/1987|  15414.5|   4.9| 5.80| 0.009  | III| 86.8(139.7)
DMG  |33.2160|115.8080|04/25/1957|215738.7|  -0.3| 5.20| 0.006  | II  | 87.0(140.1)
PAS  |33.9980|116.6060|07/08/1986|  92044.5|  11.7| 5.60| 0.008  | III| 87.2(140.3)
DMG  |31.7500|116.5000|04/29/1935|20 8 0.0|   0.0| 5.00| 0.005  | II  | 87.9(141.5)
DMG  |32.9830|115.7330|01/24/1951|  717 2.6|   0.0| 5.60| 0.008  | II  | 88.3(142.1)
DMG  |32.9500|115.7170|06/14/1953|  41729.9|   0.0| 5.50| 0.007  | II  | 89.0(143.3)
DMG  |33.9330|116.3830|12/04/1948|234317.0|   0.0| 6.50| 0.016  | IV  | 89.6(144.2)
DMG  |34.1000|116.8000|10/24/1935|1448 7.6|   0.0| 5.10| 0.005  | II  | 89.8(144.4)
DMG  |32.9000|115.7000|10/02/1928|19 1 0.0|   0.0| 5.00| 0.005  | II  | 89.9(144.6)
MGI  |34.0000|118.0000|12/25/1903|1745 0.0|   0.0| 5.00| 0.005  | II  | 90.1(145.0)
DMG  |33.8500|118.2670|03/11/1933|1425 0.0|   0.0| 5.00| 0.005  | II  | 90.4(145.4)
GSP  |33.8760|116.2670|06/29/1992|160142.8|   1.0| 5.20| 0.005  | II  | 90.4(145.5)
DMG  |34.0170|116.5000|07/25/1947|  04631.0|   0.0| 5.00| 0.005  |  I  | 91.1(146.6)
DMG  |34.0170|116.5000|07/25/1947|  61949.0|   0.0| 5.20| 0.005  | II  | 91.1(146.6)
DMG  |34.0170|116.5000|07/26/1947|  24941.0|   0.0| 5.10| 0.005  | II  | 91.1(146.6)
DMG  |34.0170|116.5000|07/24/1947|221046.0|   0.0| 5.50| 0.007  | II  | 91.1(146.6)
GSP  |33.9020|116.2840|07/24/1992|181436.2|   9.0| 5.00| 0.004  |  I  | 91.2(146.8)
DMG  |34.1000|116.7000|02/07/1889|  520 0.0|   0.0| 5.30| 0.006  | II  | 91.6(147.4)
                                         Page 3
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DMG |33.2330|115.7170|10/22/1942| 15038.0|  0.0| 5.50| 0.007 | II  | 92.4(148.7)
GSP |34.1400|117.7000|02/28/1990|234336.6|  5.0| 5.20| 0.005 | II  | 92.4(148.8)
DMG |31.7960|116.2690|06/11/1963|152338.3| -2.0| 5.80| 0.008 | III | 92.8(149.4)
GSP |34.1630|116.8550|06/28/1992|144321.0|  6.0| 5.30| 0.006 | II  | 93.1(149.8)
DMG |34.2000|117.1000|09/20/1907| 154 0.0|  0.0| 6.00| 0.010 | III | 93.3(150.1)
DMG |34.2000|117.4000|07/22/1899| 046 0.0|  0.0| 5.50| 0.006 | II  | 93.3(150.1)
GSP |33.9610|116.3180|04/23/1992|045023.0| 12.0| 6.10| 0.011 | III | 93.3(150.2)
DMG |34.1800|116.9200|01/16/1930| 02433.9|  0.0| 5.20| 0.005 | II  | 93.4(150.3)
DMG |34.1800|116.9200|01/16/1930| 034 3.6|  0.0| 5.10| 0.005 | II  | 93.4(150.3)
T-A |33.5000|115.8200|05/00/1868| 0 0 0.0|  0.0| 6.30| 0.012 | III | 93.8(151.0)



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                         EARTHQUAKE SEARCH RESULTS
                         -------------------------

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-------------------------------------------------------------------------------
    |       |        |          |  TIME   |     |     |  SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC)  |DEPTH|QUAKE|  ACC.  | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec| (km)| MAG.|   g    |INT.|  mi   [km]
----+-------+--------+----------+--------+-----+-----+--------+----+-----------
PAS |33.0980|115.6320|04/26/1981|12 928.4|  3.8| 5.70| 0.007 | II  | 95.1(153.1)
GSP |34.1950|116.8620|08/17/1992|204152.1| 11.0| 5.30| 0.005 | II  | 95.2(153.1)
PDP |33.1600|115.6370|09/02/2005|012719.8|  9.0| 5.10| 0.004 |  I  | 95.7(154.0)
PAS |34.0610|118.0790|10/01/1987|144220.0|  9.5| 5.90| 0.008 | III | 96.0(154.5)
GSN |34.2030|116.8270|06/28/1992|150530.7|  5.0| 6.70| 0.016 | IV  | 96.2(154.8)
DMG |32.2500|115.7500|12/01/1958| 6 2 0.0|  0.0| 5.50| 0.006 | II  | 96.7(155.6)
DMG |32.2500|115.7500|12/01/1958| 32118.0|  0.0| 5.80| 0.008 | II  | 96.7(155.6)
DMG |32.2500|115.7500|12/01/1958| 350 0.0|  0.0| 5.00| 0.004 |  I  | 96.7(155.6)
GSP |34.0290|116.3210|08/21/1993|014638.4|  9.0| 5.00| 0.004 |  I  | 97.1(156.2)
PAS |34.0730|118.0980|10/04/1987|105938.2|  8.2| 5.30| 0.005 | II  | 97.3(156.6)
GSG |31.8060|116.1280|03/23/1994|025916.2| 22.0| 5.00| 0.004 |  I  | 97.6(157.0)
T-A |34.0000|118.2500|01/10/1856| 0 0 0.0|  0.0| 5.00| 0.004 |  I  | 97.9(157.5)
T-A |34.0000|118.2500|03/26/1860| 0 0 0.0|  0.0| 5.00| 0.004 |  I  | 97.9(157.5)
T-A |34.0000|118.2500|09/23/1827| 0 0 0.0|  0.0| 5.00| 0.004 |  I  | 97.9(157.5)
GSP |34.0640|116.3610|09/15/1992|084711.3|  9.0| 5.20| 0.005 | II  | 97.9(157.5)
GSP |34.2390|116.8370|07/09/1992|014357.6|  0.0| 5.30| 0.005 | II  | 98.4(158.4)
PAS |33.0140|115.5550|10/16/1979| 65842.8|  9.1| 5.50| 0.006 | II  | 98.8(158.9)
DMG |34.2670|116.9670|08/29/1943| 34513.0|  0.0| 5.50| 0.006 | II  | 98.8(159.0)
DMG |34.0670|116.3330|05/18/1940| 72132.7|  0.0| 5.00| 0.004 |  I  | 98.9(159.2)
DMG |34.0670|116.3330|05/18/1940| 55120.2|  0.0| 5.20| 0.005 |  I  | 98.9(159.2)
MGI |34.1000|118.1000|07/11/1855| 415 0.0|  0.0| 6.30| 0.011 | III | 99.0(159.2)
DMG |31.8000|116.1000|10/10/1953|1849 6.0|  0.0| 5.00| 0.004 |  I  | 99.0(159.3)
DMG |33.1170|115.5670|07/29/1950|143632.0|  0.0| 5.50| 0.006 | II  | 99.1(159.4)
DMG |33.1170|115.5670|07/28/1950|175048.0|  0.0| 5.40| 0.005 | II  | 99.1(159.4)
DMG |34.2700|117.5400|09/12/1970|143053.0|  8.0| 5.40| 0.005 | II  | 99.1(159.5)
PAS |32.9270|115.5400|10/16/1979| 54910.2| 10.4| 5.10| 0.004 |  I  | 99.2(159.6)
GSP |34.1080|116.4040|06/29/1992|141338.8|  9.0| 5.40| 0.005 | II  | 99.3(159.7)
PAS |32.9280|115.5390|10/16/1979| 61948.7|  9.2| 5.10| 0.004 |  I  | 99.3(159.7)
MGI |34.0000|118.3000|09/03/1905| 540 0.0|  0.0| 5.30| 0.005 | II  | 99.6(160.3)

*******************************************************************************
-END OF SEARCH-   135 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH:   1800  TO  2006

LENGTH OF SEARCH TIME:   207  years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 9.4 MILES (15.2 km) AWAY.
```

```
LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.387 g

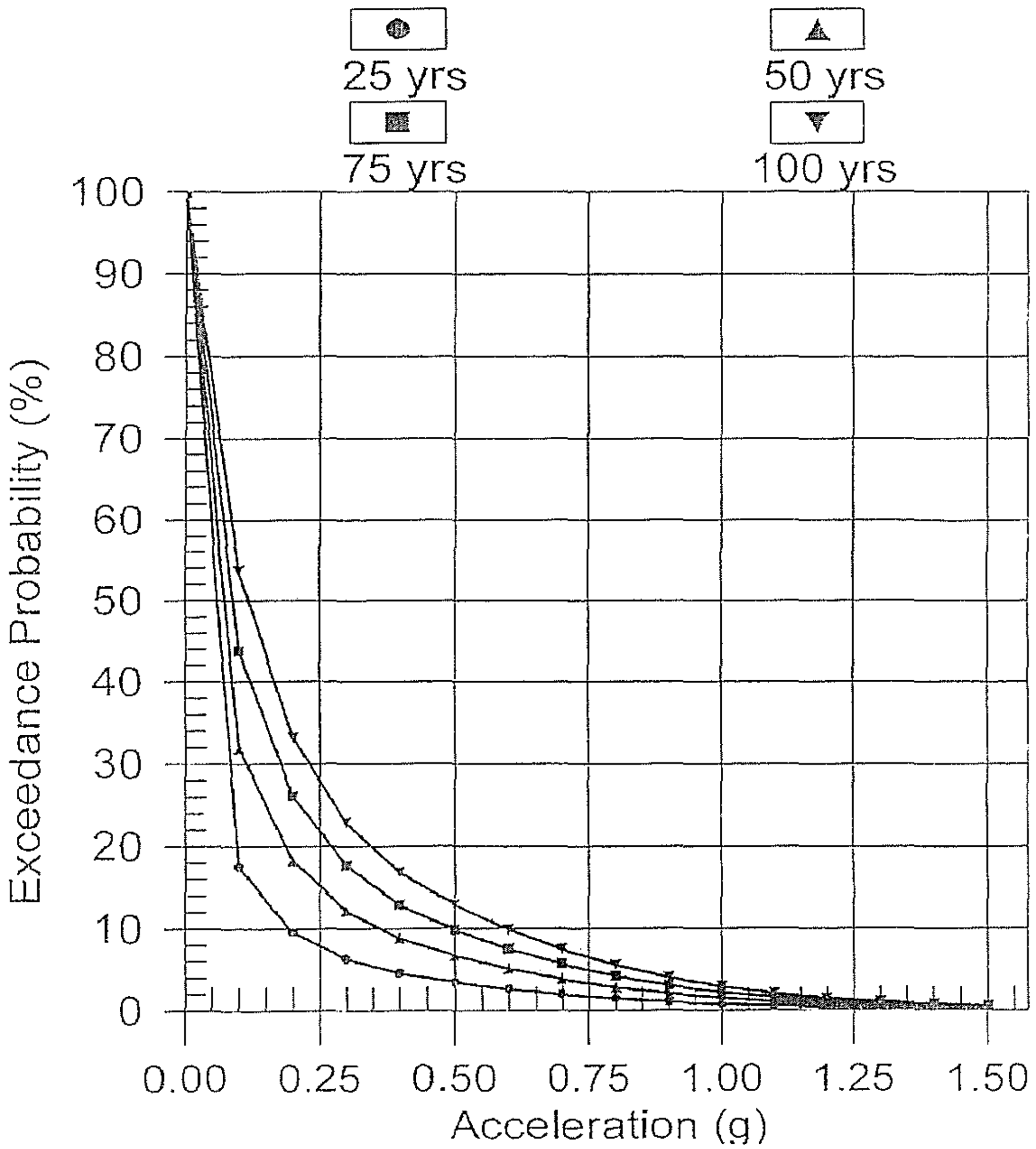
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  1.592
  b-value=  0.405
  beta-value=  0.933


-----------------------------------
TABLE OF MAGNITUDES AND EXCEEDANCES:
-----------------------------------

  Earthquake | Number of Times | Cumulative
   Magnitude |    Exceeded     | No. / Year
  -----------+-----------------+-----------
     4.0     |       135       |  0.65534
     4.5     |       135       |  0.65534
     5.0     |       135       |  0.65534
     5.5     |        48       |  0.23301
     6.0     |        22       |  0.10680
     6.5     |         7       |  0.03398
     7.0     |         1       |  0.00485
```

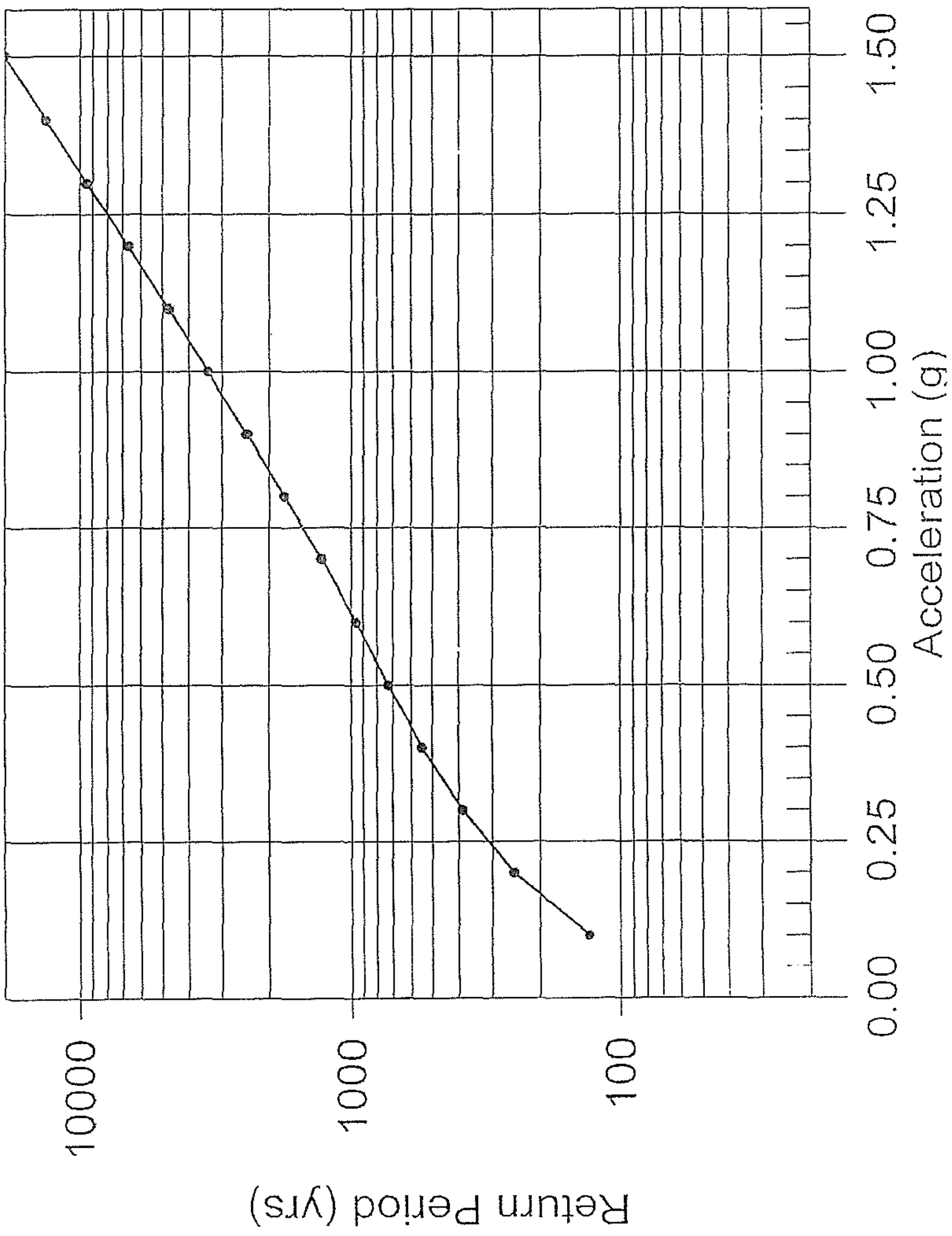
PROBABILITY OF EXCEEDANCE

SADIGH ET AL. (1997) ROCK 1



RETURN PERIOD VS. ACCELERATION

SADIGH ET AL. (1997) ROCK 1



RETURN PERIOD vs. ACCELERATION

BOZ. ET AL.(1999)HOR SR COR 1

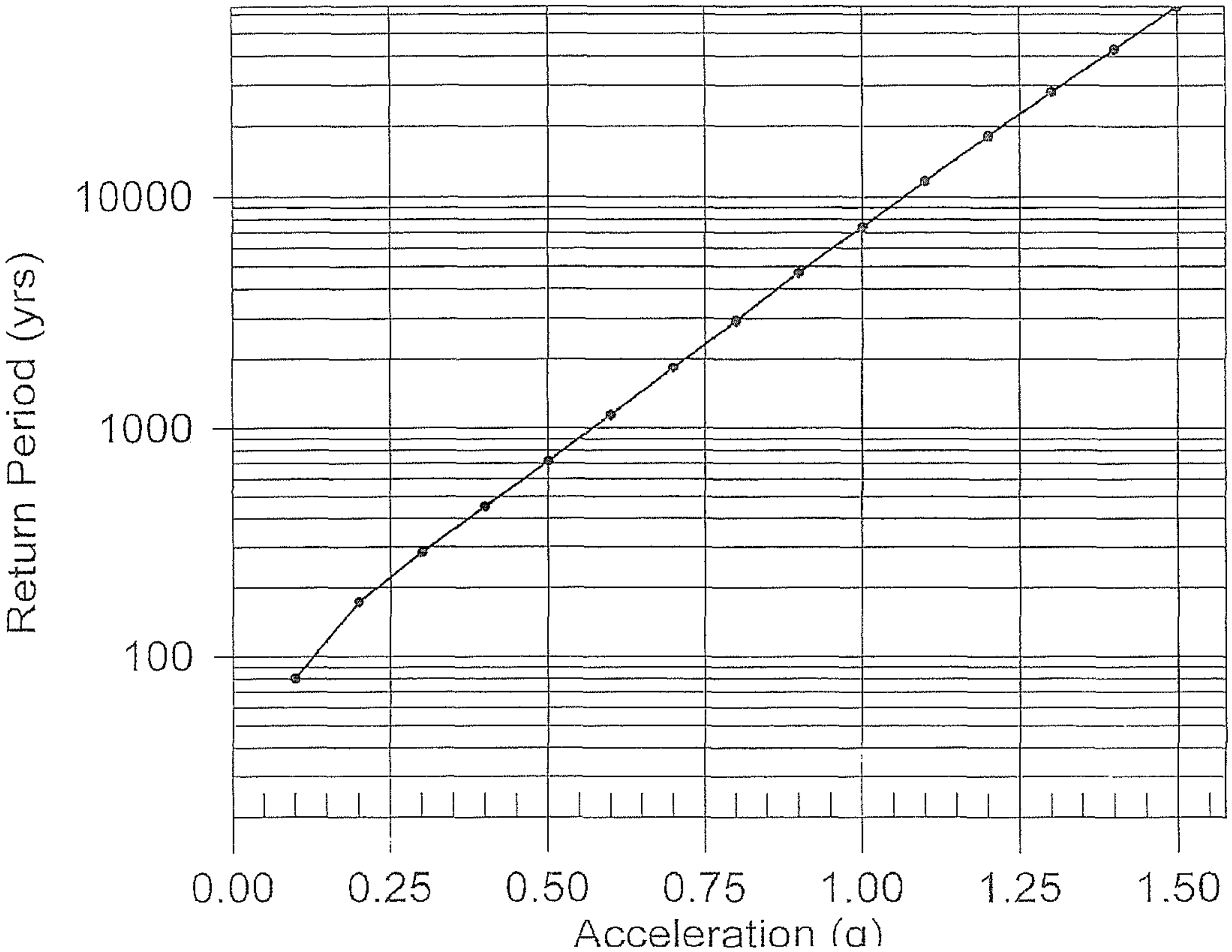


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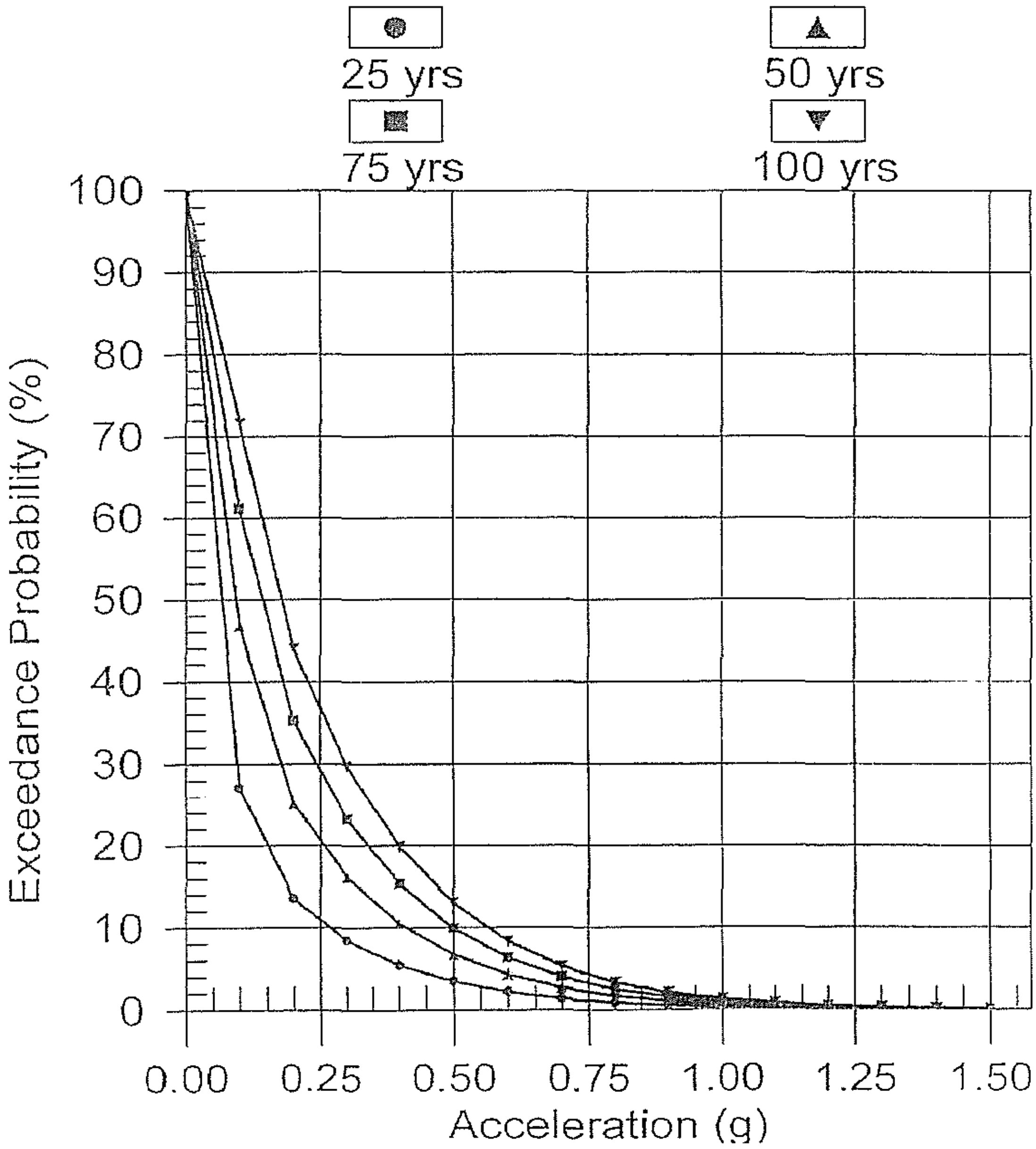
Plate C-9

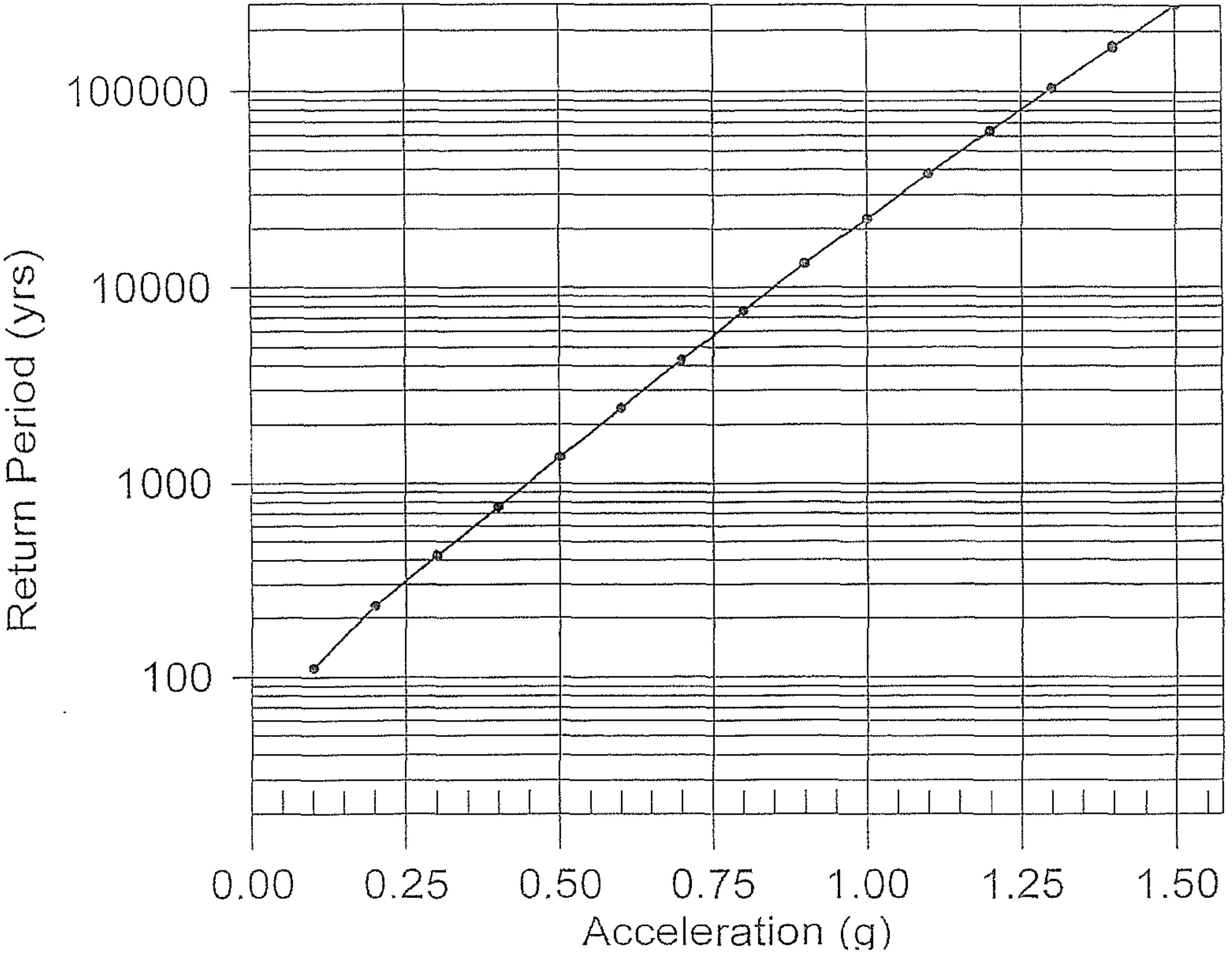
PROBABILITY OF EXCEEDANCE

BOZ. ET AL.(1999)HOR SR COR 1



RETURN PERIOD vs. ACCELERATION

BOZ. ET AL.(1999)HOR SR COR 2



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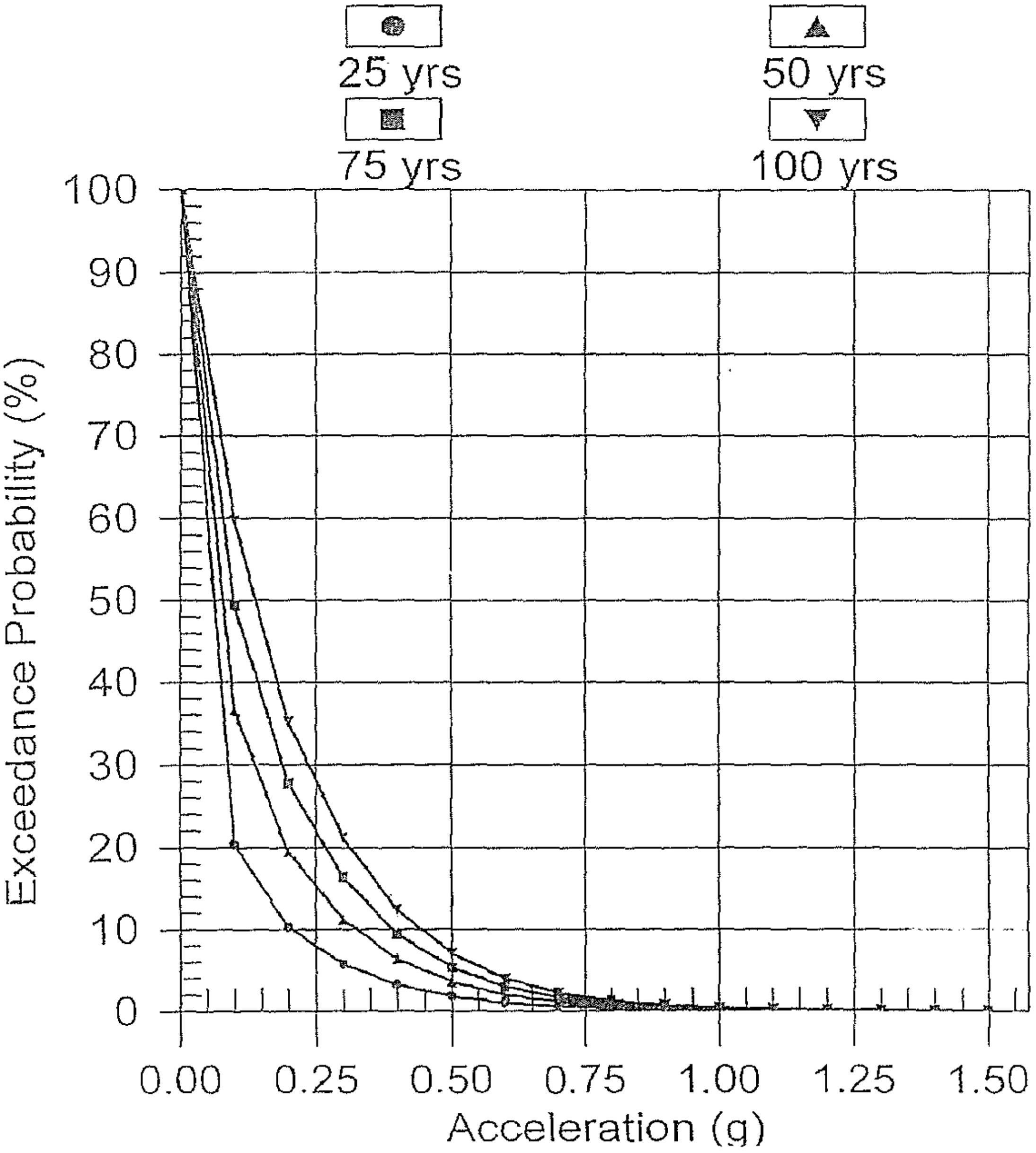
GeoSoils, Inc.

Plate C-11

PROBABILITY OF EXCEEDANCE

BOZ. ET AL.(1999)HOR SR COR 2



APPENDIX D

SLOPE STABILITY ANALYSIS

SLOPE STABILITY ANALYSIS

INTRODUCTION OF GSTABL7 v.2 COMPUTER PROGRAM

Introduction

GSTABL7 v.2 is a fully integrated slope stability analysis program. It permits the engineer to develop the slope geometry interactively and perform slope analysis from within a single program. The slope analysis portion of GSTABL7 v.2 uses a modified version of the popular STABL program, originally developed at Purdue University.

GSTABL7 v.2 performs a two dimensional limit equilibrium analysis to compute the factor of safety for a layered slope using the simplified Bishop or Janbu methods. This program can be used to search for the most critical surface or the factor of safety may be determined for specific surfaces. GSTABL7, Version 2, is programmed to handle:

1. Heterogenous soil systems
2. Anisotropic soil strength properties
3. Reinforced slopes
4. Nonlinear Mohr-Coulomb strength envelope
5. Pore water pressures for effective stress analysis using:
    a. Phreatic and piezometric surfaces
    b. Pore pressure grid
    c. R factor
    d. Constant pore water pressure
6. Pseudo-static earthquake loading
7. Surcharge boundary loads
8. Automatic generation and analysis of an unlimited number of circular, noncircular and block-shaped failure surfaces
9. Analysis of right-facing slopes
10. Both SI and Imperial units

General Information

If the reviewer wishes to obtain more information concerning slope stability analysis, the following publications may be consulted initially:

1. The Stability of Slopes, by E.N. Bromhead, Surrey University Press, Chapman and Hall, N.Y., 411 pages, ISBN 412 01061 5, 1992.

2. Rock Slope Engineering, by E. Hoek and J.W. Bray, Inst. of Mining and Metallurgy, London, England, Third Edition, 358 pages, ISNB 0 900488 573, 1981.

3. Landslides: Analysis and Control, by R.L. Schuster and R.J. Krizek (editors), Special Report 176, Transportation Research Board, National Academy of Sciences, 234 pages, ISBN 0 309 02804 3, 1978.

GSTABL7 v.2 Features

The present version of GSTABL7 v.2 contains the following features:

1. Allows user to calculate factors of safety for static stability and dynamic stability situations.

2. Allows user to analyze stability situations with different failure modes.

3. Allows user to edit input for slope geometry and calculate corresponding factor of safety.

4. Allows user to readily review on-screen the input slope geometry.

5. Allows user to automatically generate and analyze unlimited number of circular, non-circular and block-shaped failure surfaces (i.e., bedding plane, slide plane, etc.).

Input Data

Input data includes the following items:

1. Unit weight, residual cohesion, residual friction angle, peak cohesion, and peak friction angle of fill material, bedding plane, and bedrock, respectively. Residual cohesion and friction angle is used for static stability analysis, where as peak cohesion and friction angle is for dynamic stability analysis.

2. Slope geometry and surcharge boundary loads.

3. Apparent dip of bedding plane can be specified in angular range (i.e., from 0 to 90 degrees.

4. Pseudo-static earthquake loading (an earthquake loading of 0.15 *i* was used in the analysis).

Seismic Discussion

Seismic stability analyses were approximated using a pseudo-static approach. The major difficulty in the pseudo-static approach arises from the appropriate selection of the seismic coefficient used in the analysis. The use of a static inertia force equal to this acceleration during an earthquake (rigid-body response) would be extremely conservative for several reasons including: (1) only low height, stiff/dense embankments or embankments in confined areas may respond essentially as rigid structures; (2) an earthquake's inertia force is enacted on a mass for a short time period. Therefore, replacing a transient force by a pseudo-static force representing the maximum acceleration is considered unrealistic;

(3) assuming that total pseudo-static loading is applied evenly throughout the embankment for an extended period of time is an incorrect assumption, as the length of the failure surface analyzed is usually much greater than the wave length of seismic waves generated by earthquakes; and (4) the seismic waves would place portions of the mass in compression and some in tension, resulting in only a limited portion of the failure surface analyzed moving in a downslope direction, at any one instant of time.

The coefficients usually suggested by regulating agencies, counties and municipalities are in the range of 0.05g to 0.25g. For example, past regulatory guidelines within the city and county of Los Angeles indicated that the slope stability pseudostatic coefficient = 0.15 *i*.

The method developed by Krinitzsky, Gould, and Edinger (1993) which was in turn based on Taniguchi and Sasaki, 1986 (T&S, 1986), was referenced. This method is based on empirical data and the performance of existing earth embankments during seismic loading. Our review of "Guidelines for Evaluating and Mitigating Seismic Hazards in California (California Department of Conservation, Division of Mines and Geology, 1997) indicates the State of California recommends using pseudo-static coefficient of 0.15 for design earthquakes of M 8.25 or greater and using 0.1 for earthquake parameter M 6.5. Therefore, for conservatism a seismic coefficient of 0.15 *i* was used in our analysis.

Output Information

Output information includes:

1. All input data.
2. Factors of safety for the ten most critical surfaces for static and pseudo-static stability situation.
3. High quality plots can be generated. The plots include the slope geometry, the critical surfaces and the factor of safety.
4. Note, that in the analysis, a minimum of 100 trial surfaces were analyzed for each section for either static or pseudo-static analyses.

Results of Slope Stability Calculation

Cross Section A-A' was prepared for the stability analyses and represents the highest slope within the proposed project. This section is depicted in the stability analyses, Figures D-1 through D-8. Table D-1 shows parameters used in slope stability calculations. Summaries of the slope stability analysis are presented in Table D-2. Surficial slope stability calculations are presented on Figure D-9.

TABLE D-1

SOIL PARAMETERS USED

SOIL MATERIALS	PEAK VALUES	
	C (psf)	Φ (degrees)
Compacted Fill	400	25
Tertiary Ardath Formation	850	32

TABLE D-2

SUMMARY OF SLOPE ANALYSIS

STABILITY	SLOPE CONFIGURATION	SLOPE GRADIENT	FACTORS OF SAFETY		REMARKS
			STATIC	SEISMIC	
Gross	West Facing Slope	~1.8	2.35	1.22	Bishop, circular

W.O. 4971-A-SC

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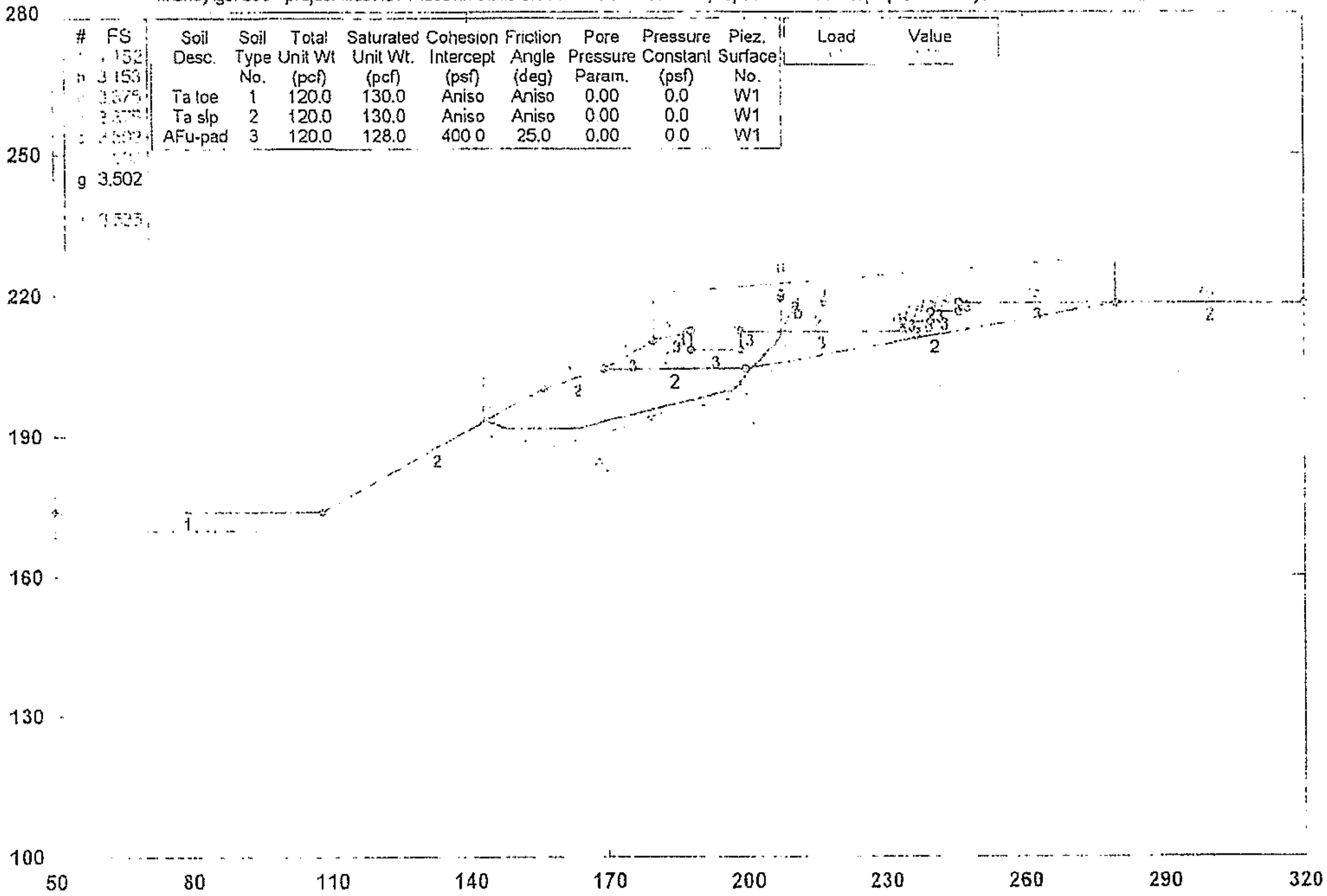
Plate D-1

Tassviri 4971 B Sec 1-1' seis mid slope Pad w/ Piers & proposed Afu cut

x:\andy\gsi 2007 project files\4971 tassviri\static\ord1s1 dash 1 static w proposed afu mid slope.pl2 Run By: AG 6/15/2007 02:09PM



#	FS
[illegible]	152
h	3 153
[illegible]	3.375
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
g	3.502
[illegible]	3.523

Soil Desc.	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.
Ta toe	1	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
Ta slp	2	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
AFu-pad	3	120.0	128.0	400 0	25.0	0.00	0 0	W1

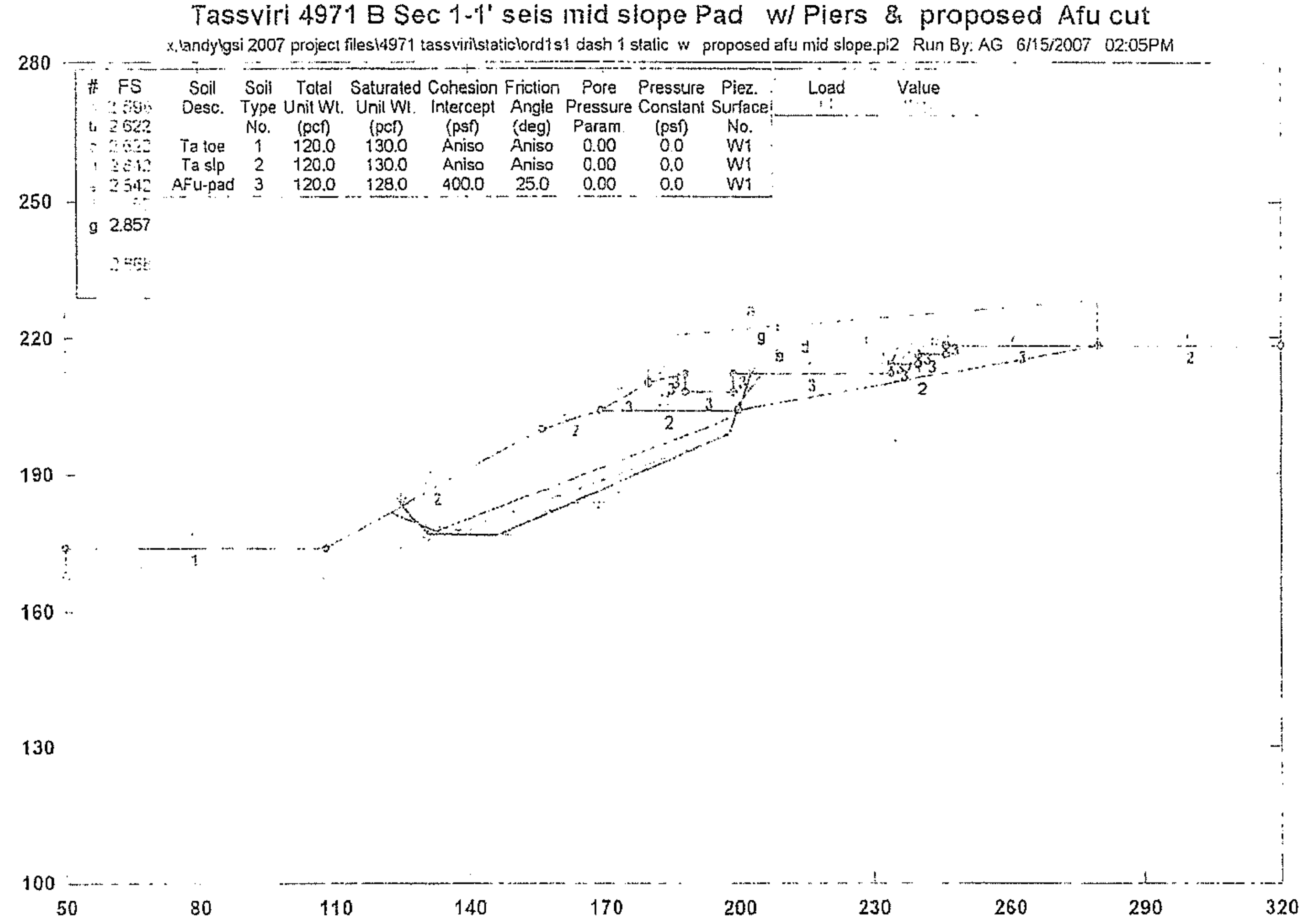
Load	Value

GSTABL7 v.2 FSmin=3.152

Safety Factors Are Calculated By The Simplified Janbu Method

W.O. 4971-A-SC

GeoSoils, Inc.



Soil Desc.	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.
Ta toe	1	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
Ta slp	2	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
AFu-pad	3	120.0	128.0	400.0	25.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=2.596

Safety Factors Are Calculated By The Simplified Janbu Method

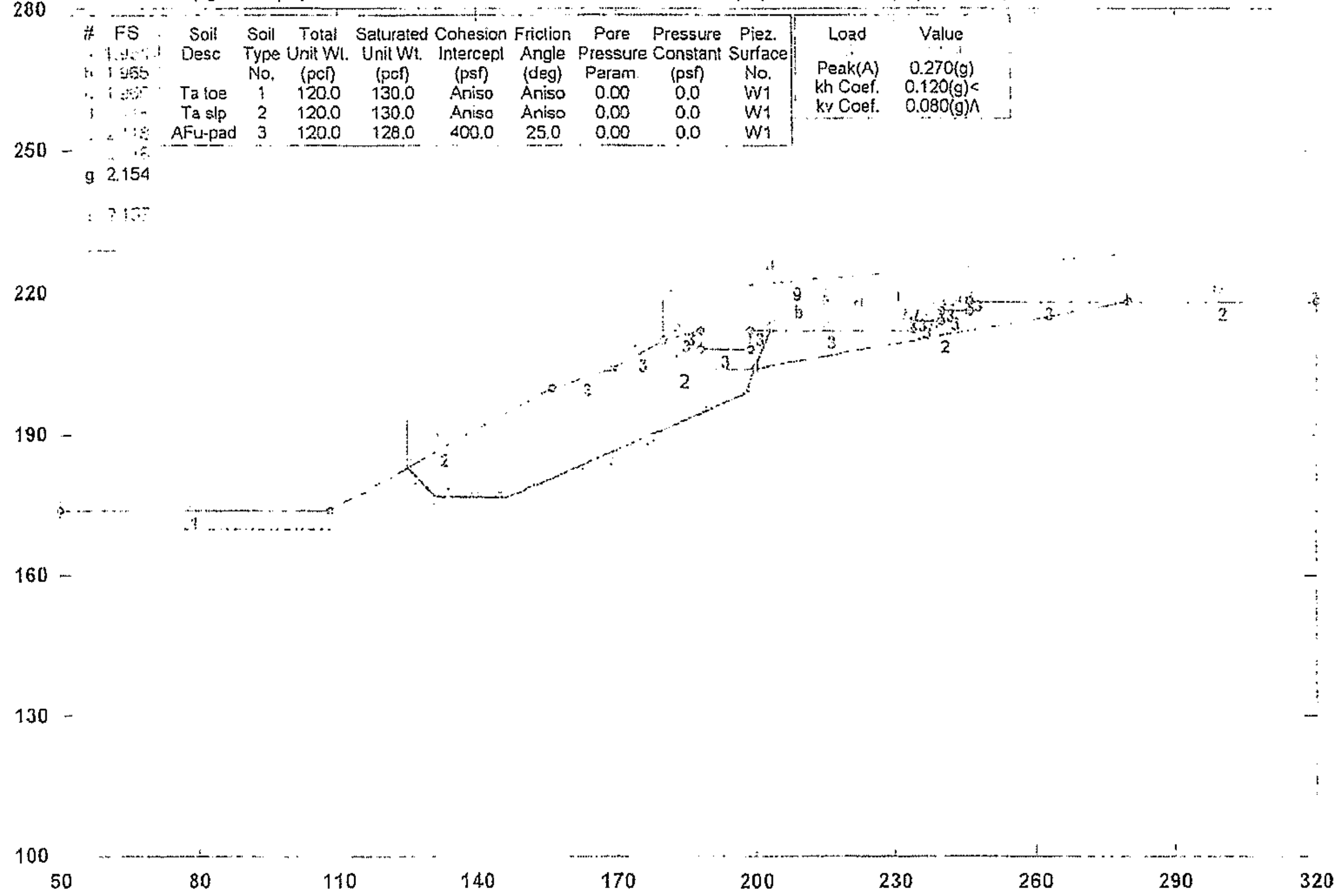
Plate D-2

W.O. 4971-A-SC

GeoSoils, Inc.

Tassviri 4971 B Sec 1-1' seis mid slope Pad w/ Piers & proposed Afu cut

x:\andy\gsi 2007 project files\4971 tassviri\seismic\ord1s1 dash 1 seismic w proposed afu mid slope pl2 Run By: AG 6/15/2007 02:04PM



Soil Desc	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.
Ta toe	1	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
Ta slp	2	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
AFu-pad	3	120.0	128.0	400.0	25.0	0.00	0.0	W1

Load	Value
Peak(A)	0.270(g)
kh Coef.	0.120(g)<
kv Coef.	0.080(g)Λ

GSTABL7 v.2 FSmin=1.951
Safety Factors Are Calculated By The Simplified Janbu Method

Plate D-3

W.O. 4971-A-SC

GeoSoils, Inc.

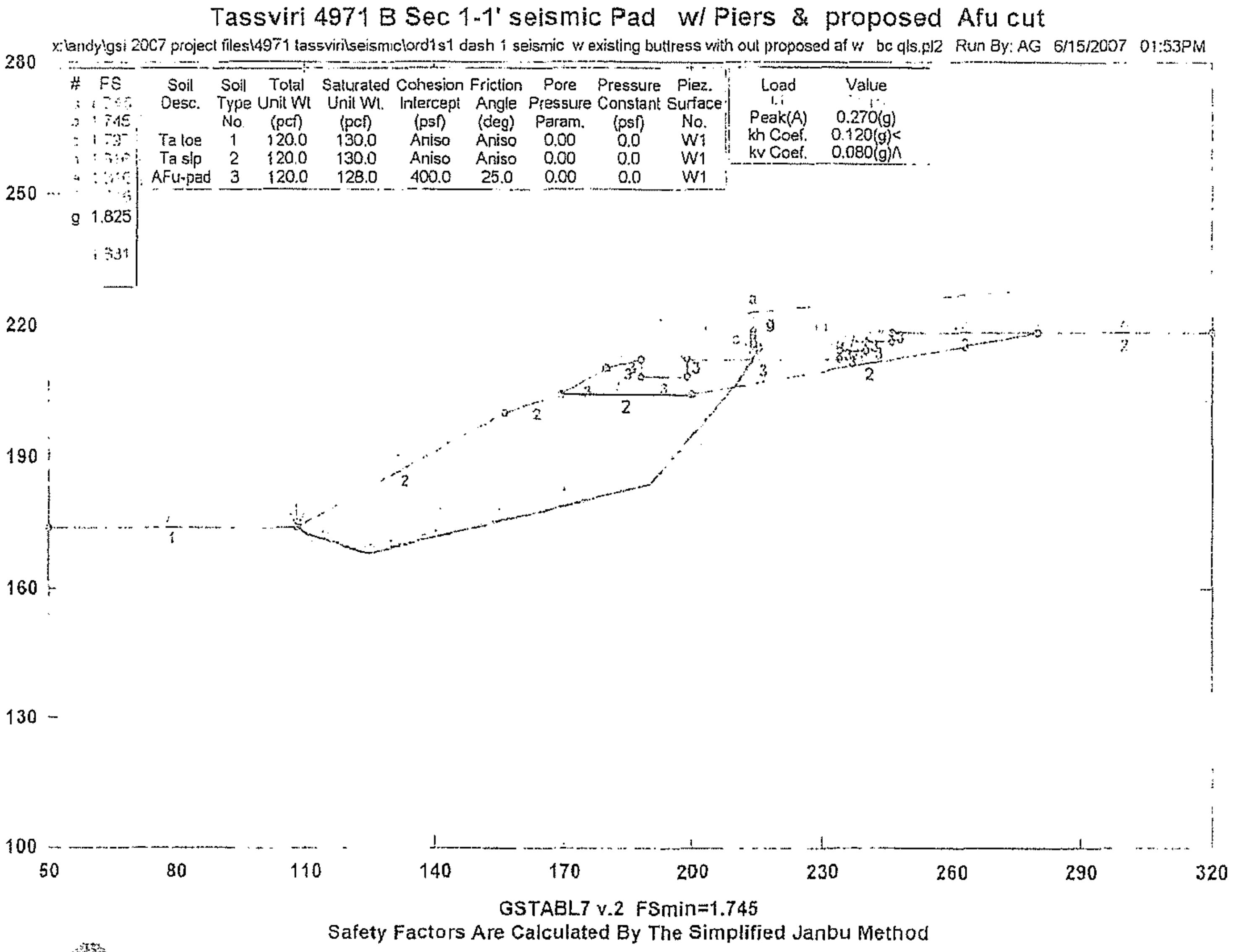


Plate D-4

W.O. 4971-A-SC

GeoSoils, Inc.

Plate D-5

Tassviri 4971 B Sec 1-1' seismic Pad w/ Piers & proposed Afu cut

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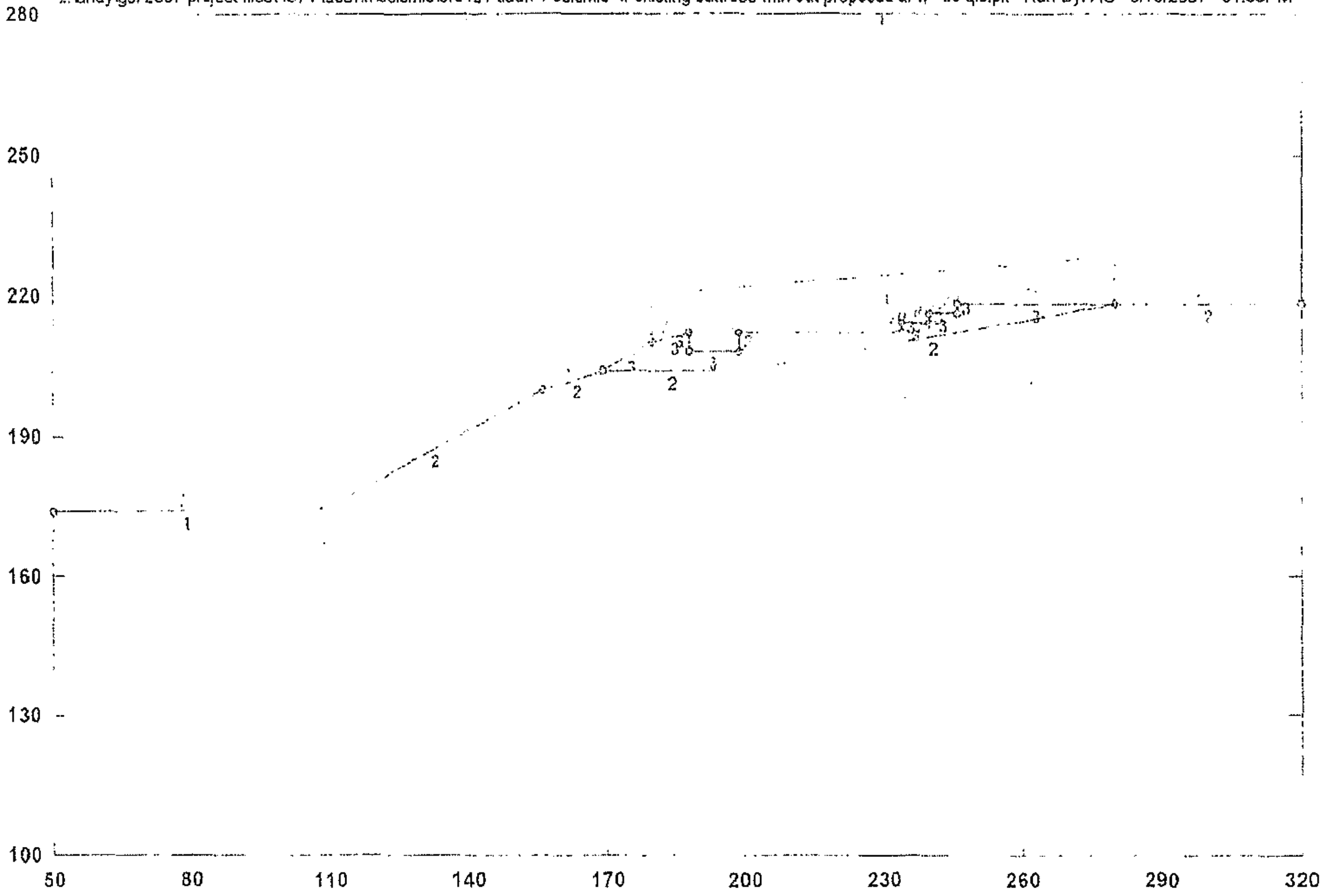

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GeoSoils, Inc.

Plate D-6

Tassviri 4971 B Sec 1-1' seismic Pad w/ Piers & proposed Afu cut

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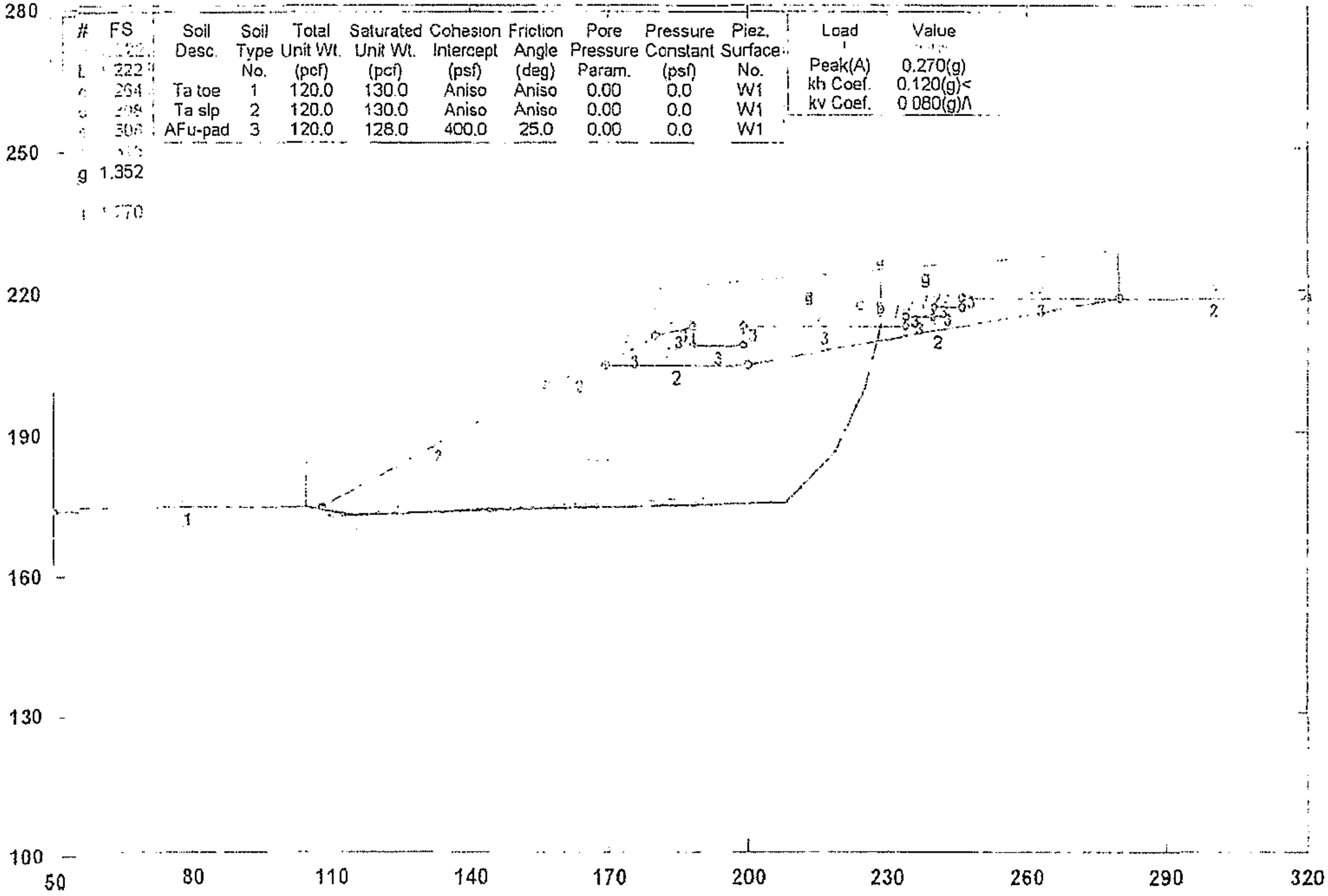

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Ta toe	1	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
Ta slp	2	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
AFu-pad	3	120.0	128.0	400.0	25.0	0.00	0.0	W1

Load	Value
Peak(A)	0.270(g)
kh Coef.	0.120(g)<
kv Coef.	0.080(g)/\

GSTABL7 v.2 FSmin=1.222

Safety Factors Are Calculated By The Simplified Janbu Method

W.O. 4971-A-SC

GeoSoils, Inc.

Plate D-7

Tassviri 4971 B Sec 1-1' static Pad w/ Piers & proposed Afu cut

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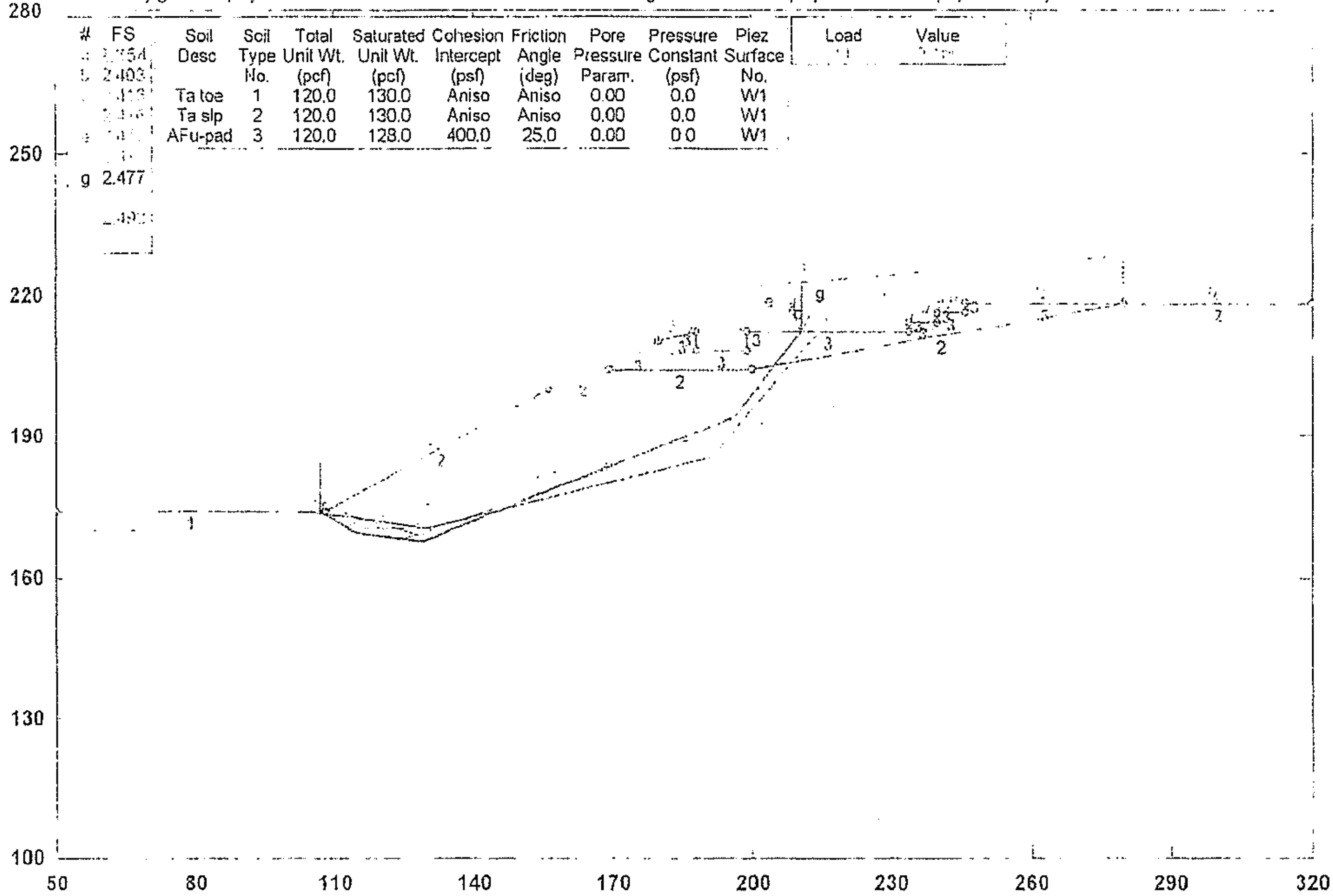

Soil Desc	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez Surface No.
Ta toe	1	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
Ta slp	2	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
AFu-pad	3	120.0	128.0	400.0	25.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=2.354

Safety Factors Are Calculated By The Simplified Janbu Method

W.O. 4971-A-SC

GeoSoils, Inc.

Tassviri 4971 B Sec 1-1' static Pad w/ Piers & proposed Afu cut

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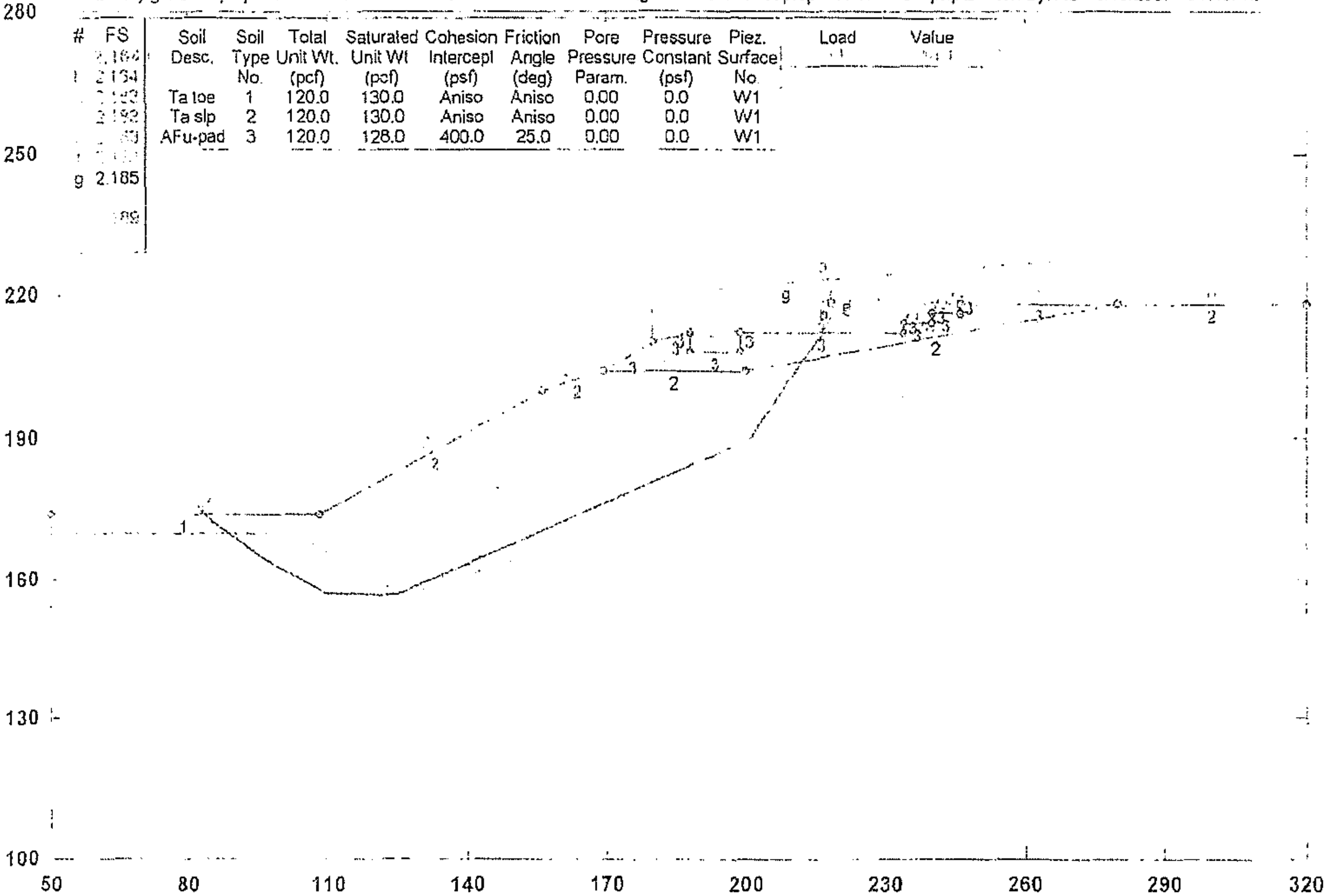
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Ta toe	1	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
Ta slp	2	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
AFu-pad	3	120.0	128.0	400.0	25.0	0.00	0.0	W1

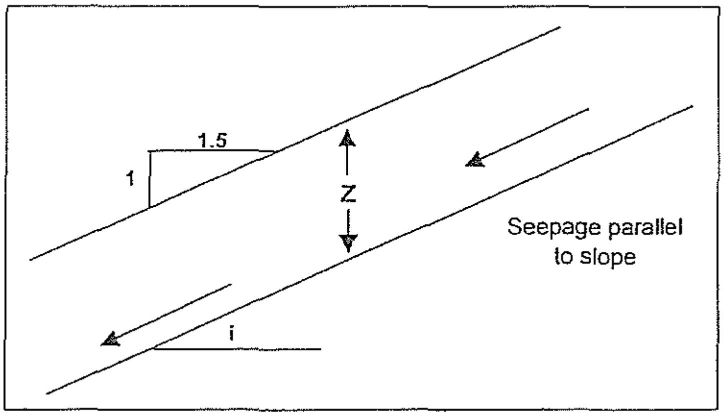


GSTABL7 v.2 FSmin=2.164

Safety Factors Are Calculated By The Simplified Janbu Method

Plate D-8

SURFICIAL SLOPE STABILITY ANALYSIS



Tract/Project:	2585 Calle De Oro
Material Type:	Silty/Sandy Claystone

Depth of Saturation (z)	4	feet
Slope Angle (i) (for 1.5:1 slopes)	33.7	degrees
Unit Weight of Water (γ_w)	62.4	lb/ft^3
Saturated Unit Weight of Soil (γ_{sat})	128	lb/ft^3
Apparent Angle of Internal Friction (ϕ)	25	degrees
Apparent Cohesion (C)	275	lb/ft^2

$$Fs = \text{Static Safety Factor} = \frac{z\,(\gamma_{sat}-\gamma_w)\,\text{Cos}^2(i)\,\text{Tan}\,(\phi) + C}{z\,(\gamma_{sat})\,\text{Sin}\,(i)\,\text{Cos}\,(i)}$$

DEPTH OF SATURATION	SLOPE	FACTOR OF SAFETY
4 FEET	1½: 1	1.52

GeoSoils, Inc.

W.O. 4971-A-SC

SURFICIAL SLOPE STABILITY

1½: 1 SLOPE

Figure D-9

APPENDIX E

GENERAL EARTHWORK AND GRADING GUIDELINES

GENERAL EARTHWORK AND GRADING GUIDELINES

General

These guidelines present general procedures and requirements for earthwork and grading as shown on the approved grading plans, including preparation of areas to filled, placement of fill, installation of subdrains, and excavations. The recommendations contained in the geotechnical report are part of the earthwork and grading guidelines and would supercede the provisions contained hereafter in the case of conflict. Evaluations performed by the consultant during the course of grading may result in new or revised recommendations which could supercede these guidelines or the recommendations contained in the geotechnical report.

The contractor is responsible for the satisfactory completion of all earthwork in accordance with provisions of the project plans and specifications. The project soil engineer and engineering geologist (geotechnical consultant), or their representatives, should provide observation and testing services, and geotechnical consultation during the duration of the project.

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

Prior to the commencement of grading, a qualified geotechnical consultant (soil engineer and engineering geologist) should be employed for the purpose of observing earthwork procedures and testing the fills for general conformance with the recommendations of the geotechnical report, the approved grading plans, and applicable grading codes and ordinances.

The geotechnical consultant should provide testing and observation so that determination may be made that the work is being accomplished as specified. It is the responsibility of the contractor to assist the consultants and keep them apprised of anticipated work schedules and changes, so that they may schedule their personnel accordingly.

All remedial removals, clean-outs, prepared ground to receive fill, key excavations, and subdrain installation should be observed and documented by the project engineering geologist and/or soil engineer prior to placing and fill. It is the contractor's responsibility to notify the engineering geologist and soil engineer when such areas are ready for observation.

Laboratory and Field Tests

Maximum dry density tests to determine the degree of compaction should be performed in accordance with American Standard Testing Materials test method ASTM designation D-1557. Random or representative field compaction tests should be performed in accordance with test methods ASTM designation D-1556, D-2937 or D-2922, and D-3017, at intervals of approximately ±2 feet of fill height or approximately every 1,000 cubic yards placed. These criteria would vary depending on the soil conditions and the size of the project. The location and frequency of testing would be at the discretion of the geotechnical consultant.

Contractor's Responsibility

All clearing, site preparation, and earthwork performed on the project should be conducted by the contractor, with observation by a geotechnical consultant, and staged approval by the governing agencies, as applicable. It is the contractor's responsibility to prepare the ground surface to receive the fill, to the satisfaction of the soil engineer, and to place, spread, moisture condition, mix, and compact the fill in accordance with the recommendations of the soil engineer. The contractor should also remove all non-earth material considered unsatisfactory by the soil engineer.

It is the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the earthwork in accordance with applicable grading guidelines, codes or agency ordinances, and approved grading plans. Sufficient watering apparatus and compaction equipment should be provided by the contractor with due consideration for the fill material, rate of placement, and climatic conditions. If, in the opinion of the geotechnical consultant, unsatisfactory conditions such as questionable weather, excessive oversized rock or deleterious material, insufficient support equipment, etc., are resulting in a quality of work that is not acceptable, the consultant will inform the contractor, and the contractor is expected to rectify the conditions, and if necessary, stop work until conditions are satisfactory.

During construction, the contractor shall properly grade all surfaces to maintain good drainage and prevent ponding of water. The contractor shall take remedial measures to control surface water and to prevent erosion of graded areas until such time as permanent drainage and erosion control measures have been installed.

SITE PREPARATION

All major vegetation, including brush, trees, thick grasses, organic debris, and other deleterious material, should be removed and disposed of off-site. These removals must be concluded prior to placing fill. In-place existing fill, soil, alluvium, colluvium, or rock materials, determined by the soil engineer or engineering geologist as being unsuitable, should be removed prior to any fill placement. Depending upon the soil conditions, these materials may be reused as compacted fills. Any materials incorporated as part of the compacted fills should be approved by the soil engineer.

Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, or other structures not located prior to grading, are to be removed or treated in a manner recommended by the soil engineer. Soft, dry, spongy, highly fractured, or otherwise unsuitable ground, extending to such a depth that surface processing cannot adequately improve the condition, should be overexcavated down to firm ground and approved by the soil engineer before compaction and filling operations continue. Overexcavated and processed soils, which have been properly mixed and moisture conditioned, should be re-compacted to the minimum relative compaction as specified in these guidelines.

Existing ground, which is determined to be satisfactory for support of the fills, should be scarified to a minimum depth of 6 to 8 inches, or as directed by the soil engineer. After the scarified ground is brought to optimum moisture content, or greater and mixed, the materials should be compacted as specified herein. If the scarified zone is greater than 6 to 8 inches in depth, it may be necessary to remove the excess and place the material in lifts restricted to about 6 to 8 inches in compacted thickness.

Existing ground which is not satisfactory to support compacted fill should be overexcavated as required in the geotechnical report, or by the on-site soils engineer and/or engineering geologist. Scarification, disc harrowing, or other acceptable forms of mixing should continue until the soils are broken down and free of large lumps or clods, until the working surface is reasonably uniform and free from ruts, hollows, hummocks, or other uneven features, which would inhibit compaction as described previously.

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical [h:v]), the ground should be stepped or benched. The lowest bench, which will act as a key, should be a minimum of 15 feet wide and should be at least 2 feet deep into firm material, and approved by the soil engineer and/or engineering geologist. In fill over cut slope conditions, the recommended minimum width of the lowest bench or key is also 15 feet, with the key founded on firm material, as designated by the geotechnical consultant. As a general rule, unless specifically recommended otherwise by the soil engineer, the minimum width of fill keys should be approximately equal to ½ the height of the slope.

Standard benching is generally 4 feet (minimum) vertically, exposing firm, acceptable material. Benching may be used to remove unsuitable materials, although it is understood that the vertical height of the bench may exceed 4 feet. Pre-stripping may be considered for unsuitable materials in excess of 4 feet in thickness.

All areas to receive fill, including processed areas, removal areas, and the toes of fill benches, should be observed and approved by the soil engineer and/or engineering geologist prior to placement of fill. Fills may then be properly placed and compacted until design grades (elevations) are attained.

COMPACTED FILLS

Any earth materials imported or excavated on the property may be utilized in the fill provided that each material has been determined to be suitable by the soil engineer. These materials should be free of roots, tree branches, other organic matter, or other deleterious materials. All unsuitable materials should be removed from the fill as directed by the soil engineer. Soils of poor gradation, undesirable expansion potential, or substandard strength characteristics may be designated by the consultant as unsuitable and may require blending with other soils to serve as a satisfactory fill material.

Fill materials derived from benching operations should be dispersed throughout the fill area and blended with other approved material. Benching operations should not result in the benched material being placed only within a single equipment width away from the fill/bedrock contact.

Oversized materials defined as rock, or other irreducible materials, with a maximum dimension greater than 12 inches, should not be buried or placed in fills unless the location of materials and disposal methods are specifically approved by the soil engineer. Oversized material should be taken offsite, or placed in accordance with recommendations of the soil engineer in areas designated as suitable for rock disposal. Per the UBC/CBC, oversized material should not be placed within 10 feet vertically of finish grade (elevation) or within 20 feet horizontally of slope faces (any variation will require prior approval from the governing agency).

To facilitate future trenching, rock (or oversized material) should not be placed within 10 feet from finish grade, the range of foundation excavations, future utilities, or underground construction unless specifically approved by the soil engineer and/or the developer's representative.

If import material is required for grading, representative samples of the materials to be utilized as compacted fill should be analyzed in the laboratory by the soil engineer to determine it's physical properties and suitability for use onsite. If any material other than that previously tested is encountered during grading, an appropriate analysis of this material should be conducted by the soil engineer as soon as possible.

Approved fill material should be placed in areas prepared to receive fill in near horizontal layers, that when compacted, should not exceed about 6 to 8 inches in thickness. The soil engineer may approve thick lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness. Each layer should be spread evenly and blended to attain uniformity of material and moisture suitable for compaction.

Fill layers at a moisture content less than optimum should be watered and mixed, and wet fill layers should be aerated by scarification, or should be blended with drier material. Moisture conditioning, blending, and mixing of the fill layer should continue until the fill materials have a uniform moisture content at, or above, optimum moisture.

After each layer has been evenly spread, moisture conditioned, and mixed, it should be uniformly compacted to a minimum of 90 percent of the maximum density as determined by ASTM test designation D-1557, or as otherwise recommended by the soil engineer. Compaction equipment should be adequately sized and should be specifically designed for soil compaction or of proven reliability to efficiently achieve the specified degree of compaction.

Where tests indicate that the density of any layer of fill, or portion thereof, is below the required relative compaction, or improper moisture is in evidence, the particular layer or portion shall be re-worked until the required density and/or moisture content has been attained. No additional fill shall be placed in an area until the last placed lift of fill has been tested and found to meet the density and moisture requirements, and is approved by the soil engineer.

In general, per the UBC/CBC, fill slopes should be designed and constructed at a gradient of 2:1 (h:v), or flatter. Compaction of slopes should be accomplished by over-building a minimum of 3 feet horizontally, and subsequently trimming back to the design slope configuration. Testing shall be performed as the fill is elevated to evaluate compaction as the fill core is being developed. Special efforts may be necessary to attain the specified compaction in the fill slope zone. Final slope shaping should be performed by trimming and removing loose materials with appropriate equipment. A final determination of fill slope compaction should be based on observation and/or testing of the finished slope face. Where compacted fill slopes are designed steeper than 2:1 (h:v), prior approval from the governing agency, specific material types, a higher minimum relative compaction, special reinforcement, and special grading procedures will be recommended.

If an alternative to over-building and cutting back the compacted fill slopes is selected, then special effort should be made to achieve the required compaction in the outer 10 feet of each lift of fill by undertaking the following:

1. An extra piece of equipment consisting of a heavy, short-shanked sheepsfoot should be used to roll (horizontal) parallel to the slopes continuously as fill is placed. The sheepsfoot roller should also be used to roll perpendicular to the slopes, and extend out over the slope to provide adequate compaction to the face of the slope.

2. Loose fill should not be spilled out over the face of the slope as each lift is compacted. Any loose fill spilled over a previously completed slope face should be trimmed off or be subject to re-rolling.

3. Field compaction tests will be made in the outer (horizontal) ±2 to ±8 feet of the slope at appropriate vertical intervals, subsequent to compaction operations.

4. After completion of the slope, the slope face should be shaped with a small tractor and then re-rolled with a sheepsfoot to achieve compaction to near the slope face. Subsequent to testing to evaluate compaction, the slopes should be grid-rolled to

achieve compaction to the slope face. Final testing should be used to evaluate compaction after grid rolling.

5. Where testing indicates less than adequate compaction, the contractor will be responsible to rip, water, mix, and recompact the slope material as necessary to achieve compaction. Additional testing should be performed to evaluate compaction.

6. Erosion control and drainage devices should be designed by the project civil engineer in compliance with ordinances of the controlling governmental agencies, and/or in accordance with the recommendation of the soil engineer or engineering geologist.

SUBDRAIN INSTALLATION

Subdrains should be installed in approved ground in accordance with the approximate alignment and details indicated by the geotechnical consultant. Subdrain locations or materials should not be changed or modified without approval of the geotechnical consultant. The soil engineer and/or engineering geologist may recommend and direct changes in subdrain line, grade, and drain material in the field, pending exposed conditions. The location of constructed subdrains, especially the outlets, should be recorded by the project civil engineer.

EXCAVATIONS

Excavations and cut slopes should be examined during grading by the engineering geologist. If directed by the engineering geologist, further excavations or overexcavation and refilling of cut areas should be performed, and/or remedial grading of cut slopes should be performed. When fill over cut slopes are to be graded, unless otherwise approved, the cut portion of the slope should be observed by the engineering geologist prior to placement of materials for construction of the fill portion of the slope. The engineering geologist should observe all cut slopes, and should be notified by the contractor when excavation of cut slopes commence.

If, during the course of grading, unforeseen adverse or potentially adverse geologic conditions are encountered, the engineering geologist and soil engineer should investigate, evaluate, and make appropriate recommendations for mitigation of these conditions. The need for cut slope buttressing or stabilizing should be based on in-grading evaluation by the engineering geologist, whether anticipated or not.

Unless otherwise specified in soil and geological reports, no cut slopes should be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies. Additionally, short-term stability of temporary cut slopes is the contractor's responsibility.

Erosion control and drainage devices should be designed by the project civil engineer and should be constructed in compliance with the ordinances of the controlling governmental agencies, and/or in accordance with the recommendations of the soil engineer or engineering geologist.

COMPLETION

Observation, testing, and consultation by the geotechnical consultant should be conducted during the grading operations in order to state an opinion that all cut and fill areas are graded in accordance with the approved project specifications. After completion of grading, and after the soil engineer and engineering geologist have finished their observations of the work, final reports should be submitted subject to review by the controlling governmental agencies. No further excavation or filling should be undertaken without prior notification of the soil engineer and/or engineering geologist.

All finished cut and fill slopes should be protected from erosion and/or be planted in accordance with the project specifications and/or as recommended by a landscape architect. Such protection and/or planning should be undertaken as soon as practical after completion of grading.

JOB SAFETY

General

At GSI, getting the job done safely is of primary concern. The following is the company's safety considerations for use by all employees on multi-employer construction sites. On-ground personnel are at highest risk of injury, and possible fatality, on grading and construction projects. GSI recognizes that construction activities will vary on each site, and that site safety is the prime responsibility of the contractor; however, everyone must be safety conscious and responsible at all times. To achieve our goal of avoiding accidents, cooperation between the client, the contractor, and GSI personnel must be maintained.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of field personnel on grading and construction projects:

Safety Meetings:	GSI field personnel are directed to attend contractor's regularly scheduled and documented safety meetings.
Safety Vests:	Safety vests are provided for, and are to be worn by GSI personnel, at all times, when they are working in the field.
Safety Flags:	Two safety flags are provided to GSI field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.
Flashing Lights:	All vehicles stationary in the grading area shall use rotating or flashing amber beacons, or strobe lights, on the vehicle during all field testing. While operating a vehicle in the grading area, the emergency flasher on the vehicle shall be activated.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

Test Pits Location, Orientation, and Clearance

The technician is responsible for selecting test pit locations. A primary concern should be the technician's safety. Efforts will be made to coordinate locations with the grading contractor's authorized representative, and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractor's authorized representative (supervisor, grade checker, dump man, operator, etc.) should direct excavation of the pit and safety during the test period. Of paramount concern should be the soil technician's safety, and obtaining enough tests to represent the fill.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic, whenever possible. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates the fill be maintained in a driveable condition. Alternatively, the contractor may wish to park a piece of equipment in front of the test holes, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits. No grading equipment should enter this zone during the testing procedure. The zone should extend approximately 50 feet outward from the center of the test pit. This zone is established for safety and to avoid excessive ground vibration, which typically decreases test results.

When taking slope tests, the technician should park the vehicle directly above or below the test location. If this is not possible, a prominent flag should be placed at the top of the slope. The contractor's representative should effectively keep all equipment at a safe operational distance (e.g., 50 feet) away from the slope during this testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location, well away from the equipment traffic pattern. The contractor should inform our personnel of all changes to haul roads, cut and fill areas or other factors that may affect site access and site safety.

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is required, by company policy, to immediately withdraw and notify his/her supervisor. The grading contractor's representative will be contacted in an effort to affect a solution. However, in the interim, no further testing will be performed until the situation is rectified. Any fill placed can be considered unacceptable and subject to reprocessing, recompaction, or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to the technician's attention and notify this office. Effective communication and coordination between the contractor's representative and the soil technician is strongly encouraged in order to implement the above safety plan.

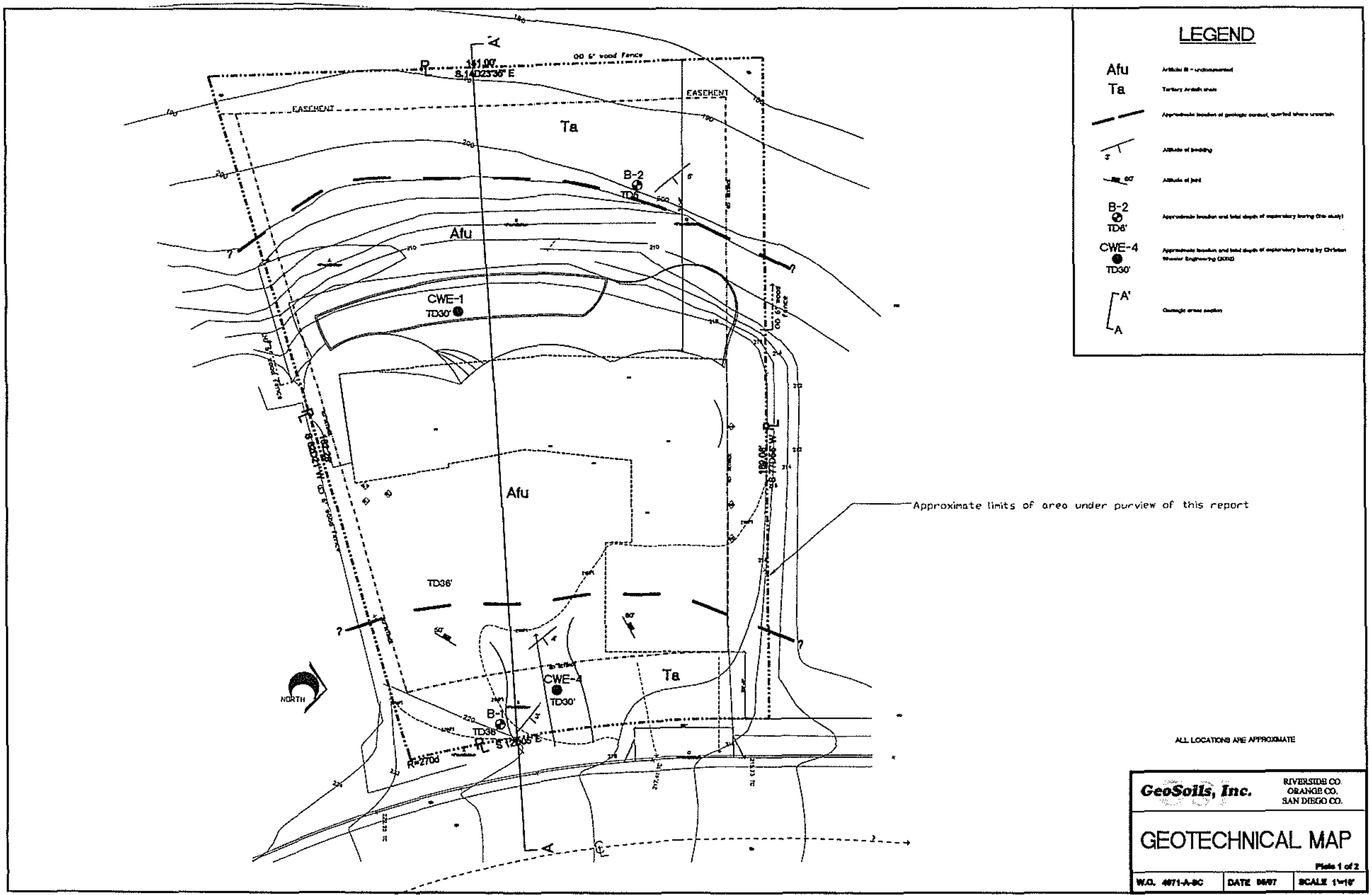
Trench and Vertical Excavation

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or other debris which could fall into the trench; or 3) displays any other evidence of any unsafe conditions regardless of depth.

All trench excavations or vertical cuts in excess of 5 feet deep, which any person enters, should be shored or laid back. Trench access should be provided in accordance with CAL-OSHA and/or state and local standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns or other reasons could be subject to reprocessing and/or removal.

If GSI personnel become aware of anyone working beneath an unsafe trench wall or vertical excavation, we have a legal obligation to put the contractor and owner/developer on notice to immediately correct the situation. If corrective steps are not taken, GSI then has an obligation to notify CAL-OSHA and/or the proper controlling authorities.



ALL LOCATIONS ARE APPROXIMATE

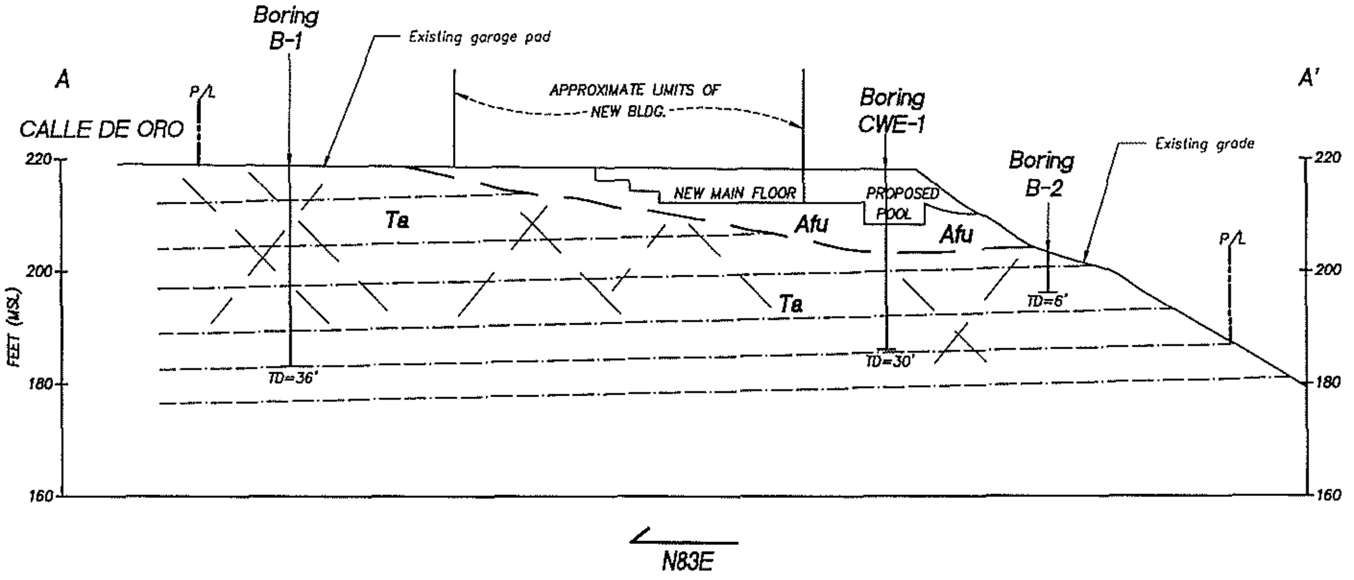
GeoSoils, Inc.

RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

GEOTECHNICAL MAP

Plate 1 of 2

W.O. 4971-A-SC | DATE 06/07 | SCALE 1"=10'



LEGEND

Afu Artificial fill - undocumented

Ta Tertiary Ardath shale

Approximate location of geologic contact

Bedding

Joint

ALL LOCATIONS ARE APPROXIMATE

GeoSoils, Inc.

RIVERSIDE CO.
ORANGE CO.
SAN DIEGO CO.

GEOLOGIC CROSS SECTION A-A'

Plate 2 of 2

W.O. 4791-A-SC	DATE 06/07	SCALE 1"=20'

Geotechnical Exploration, Inc.
SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

28 June 2016

Trevor and Staci Klein
2585 Calle del Oro
La Jolla, CA 92037

Job No. 13-10407

Subject: **Addendum Geotechnical Report Response to City Reviewer**
Klein Residence
2585 Calle del Oro
La Jolla, California

Dear Mr. and Mrs. Klein:

In accordance with your request, and as required by LDR Geology Reviewer, we are replying to comments in a memo with a completion date of October 16, 2015. The LDR Reviewer has reviewed our Limited Geotechnical Investigation report dated February 10, 2014, as well as Site Development Plan by Studio William Hefner Architecture dated September 17, 2015.

Issue No. 4: *"The geotechnical consultant indicates that based on their observations, the materials of the Ardath Shale formation are considered stable. Provide the logs of the subsurface exploration that provides the detailed direct observation and mapping of the bedding attitudes conducted by an engineering geologist. (New Issue).*

GEI Response: In our geotechnical report dated February 10, 2014, we provided boring logs of subsurface exploration at the site. The borings were performed with small diameter augers and obtained 3-inch-diameter soil samples. No large diameter borings were excavated since they were not considered necessary. We observed nearby bedrock exposures and reviewed the geological map by Kennedy and Tan (2008) that indicated bedding attitudes in this area were not unfavorable, with strikes generally N30°W to N65°W and dips northeast at angles of 3 to 5 degrees, with direction parallel to the hillside and not out of slope. We are in the process of performing supplemental test pits on the site for direct observation and mapping of the bedding attitudes.

Issue No. 5: *"The geotechnical consultant should confirm that the setback between the descending slope and outer edge of the proposed building foundations is adequate to provide protection from slope drainage, erosion and shallow failures over the expected life of the structures. (New Issue*).

GEI Response: The building foundations are anticipated to be a sufficient distance from the slope face to comply with the 8-foot minimum setback. Other improvements such as retaining walls close to or on the slope face will need to have the foundations sufficiently embedded on the slope side to comply with the minimum required setback. Adequate embedment will be confirmed by the geotechnical consultant during foundation excavation inspection. Proper foundation embedment will provide adequate protection against erosion, drainage, and shallow failures on the slope face over the expected life of the structures.

Issue No. 6: *"The plans indicate porous paving design that may result in passive infiltration. If passive infiltration may occur, incorporate an impermeable liner into the storm water best management practice design or as recommended by the geotechnical consultant. (New Issue)".*

GEI Response: It is our opinion that passive infiltration may occur, so we recommend that an impermeable liner be incorporated into the storm water BMP design.

Issue No. 7: *"The copy of the referenced geotechnical report submitted for review is missing cross section A-A'. (New Issue)".*

GEI Response: We have included a copy of cross section A-A' that was missing from the referenced geotechnical report. However, we are in the process of revising the geotechnical map and cross section based on the current building footprint.

Issue No. 8: *"Submit original quality prints and digital copies (on CD/DVD/or USB data storage device) of the referenced and requested geotechnical reports. (New Issue)".*

GEI Response: We are providing a quality copy of this report and a copy on CD.

If you have further questions regarding this letter, please contact our office. Reference to our **Job No. 13-10407** will help expedite a response to your inquiry.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Jaime A. Cerros, P.E.
R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

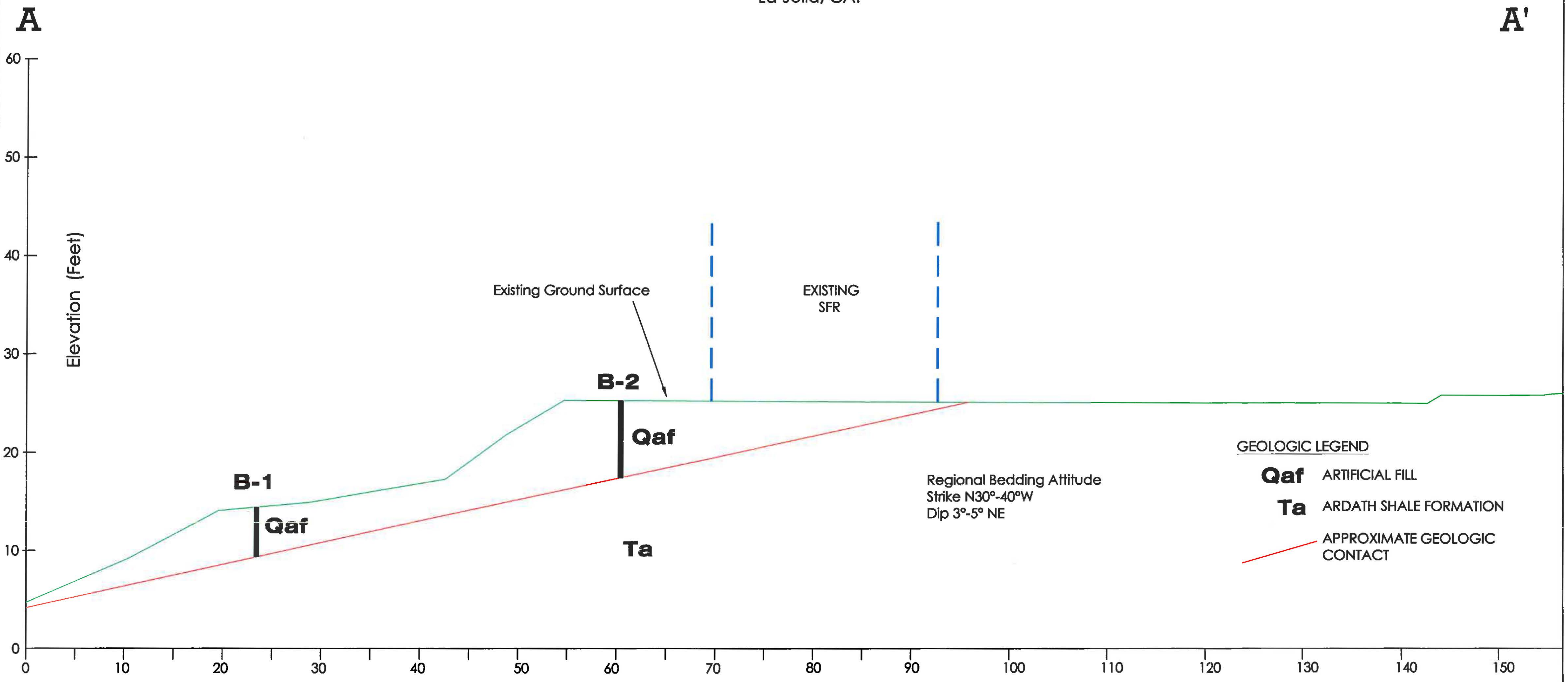
Jonathan A. Browning
C.E.G. 2615/P.G. 9012
Senior Project Geologist

REGISTERED PROFESSIONAL ENGINEER
JAIME A. CERROS
No. 002007
Exp 9/30/17
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
JONATHAN A. BROWNING
No. 2615
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Geologic Cross Section A-A'

Klein Residence Additions
2585 Calle Del Oro
La Jolla, CA.



NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

Figure No. VIa
Job No. 13-10407

Geotechnical Exploration, Inc.

February 2014

13-10407-aa

Geotechnical Exploration, Inc.

SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

04 August 2016

Trevor and Staci Klein
2585 Calle del Oro
La Jolla, CA 92037

Job No. 13-10407

Subject: **Addendum Geotechnical Report Response to City Reviewer**
Klein Residence
2585 Calle del Oro
La Jolla, California

Dear Mr. and Mrs. Klein:

In accordance with your request and as required by LDR-Geology Reviewer, we are replying to comments in a memo with a completion date of July 28, 2016 (Cycle 6). The LDR reviewer has reviewed our Limited Geotechnical Investigation report dated February 10, 2014 and our Addendum Geotechnical Report Response to City Reviewer, dated June 28 2016, as well as a Site Development Plan by Studio William Hefner Architecture dated July 1, 2016.

Issue No. 10: *Storm Water Requirements for proposed conceptual development will be evaluated by LDR-Engineering review. Priority Development Projects (PDPs) may require an investigation of storm water infiltration feasibility in accordance with the Storm Water Standards (including Appendix C and D). Check with your LDR-Engineering reviewer on requirements. LDR-Engineering may determine that LDR-Geology review of a storm water infiltration evaluation is required. (New Issue).*

GEI Response: We are in the process of performing an investigation of storm water infiltration feasibility and although infiltration may be feasible, it is our opinion that any long term infiltration may result in geotechnical hazards which cannot be reasonably mitigated to an acceptable level.

Issue No. 11: *The geotechnical consultant indicates that an addendum report providing the logs of the subsurface exploration that provides the detailed direct observation and mapping of the bedding attitudes conducted by an engineering geologist is in progress. Submit a copy of that report for review. (New Issue).*

GEI Response: In our geotechnical report dated February 10, 2014, we provided boring logs of subsurface exploration at the site. The borings were performed with small diameter augers and obtained 3-inch-diameter soil samples. No large diameter borings were excavated since they were not considered necessary for the current remodel. We observed nearby bedrock exposures and reviewed the geological map by Kennedy and Tan (2008) that indicated bedding attitudes in this area were not unfavorable, with strikes generally N30°W to N65°W and dips northeast at angles of 3 to 5 degrees, with direction parallel to the hillside and not out of slope. We confirmed the reported bedding attitudes in our supplemental test pits placed in the eastern portion of the site on July 18, 2016. We encountered the Ardath Shale Formation at a depth of 1 to 2 feet and measured strikes generally N60W and dips 3 degrees northeast. The bedding dips into the hillside and is considered to be favorable (refer to Appendix A, Slope Stability Analysis).

Issue No. 12: Submit original quality prints and digital copies (on CD/DVD/or USB data storage device) of the referenced and requested geotechnical reports for our records. (New Issue).

GEI Response: We are providing a quality copy of this report and a copy on CD.

If you have further questions regarding this letter, please contact our office. Reference to our **Job No. 13-10407** will help expedite a response to your inquiry.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Jaime A. Cerros, P.E.
R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

Jonathan A. Browning
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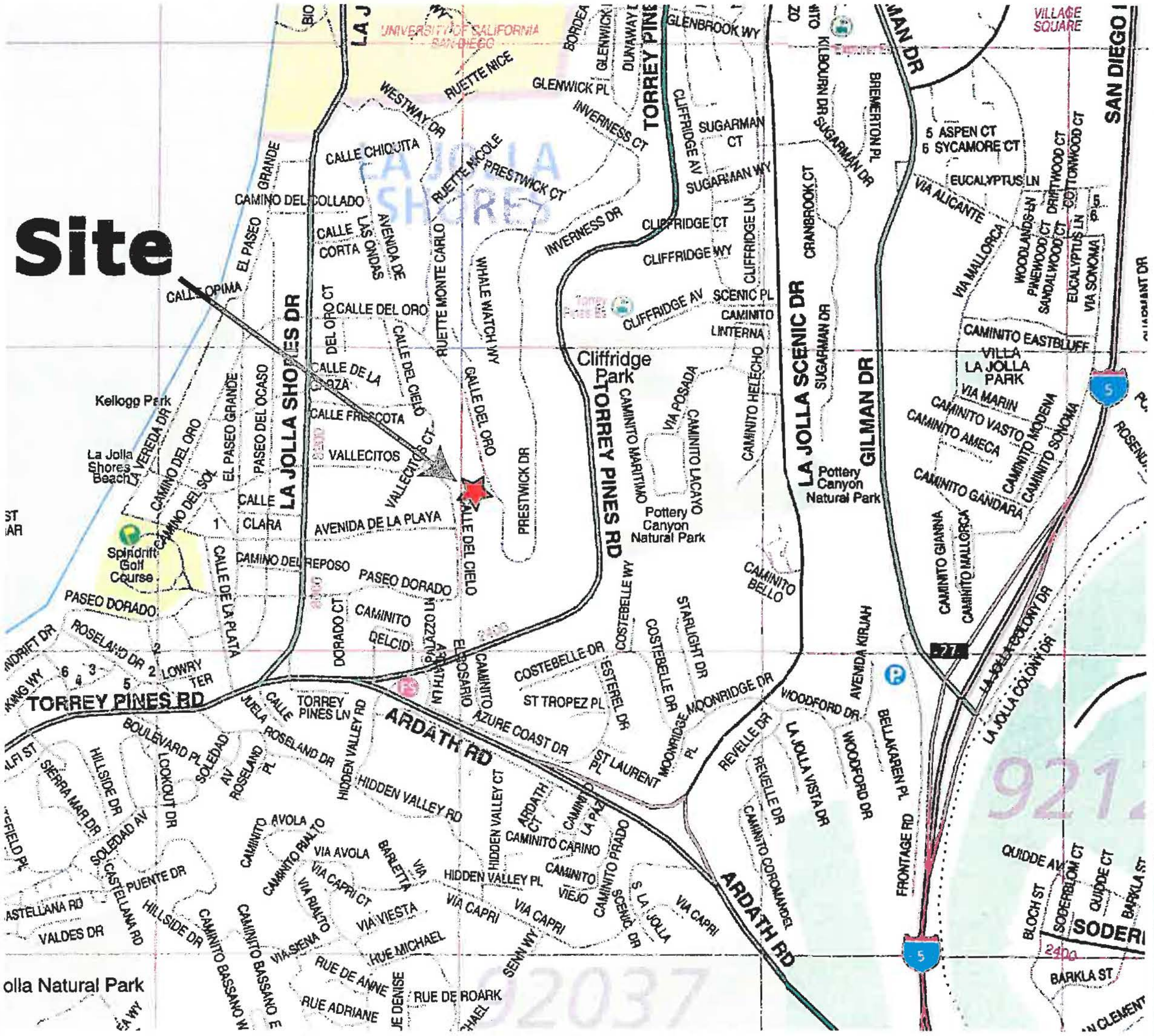
Senior Project Geologist

JAIME A. CERROS
No. 002007
Exp 9/30/17

JONATHAN A. BROWNING
No. 2615
CERTIFIED ENGINEERING GEOLOGIST

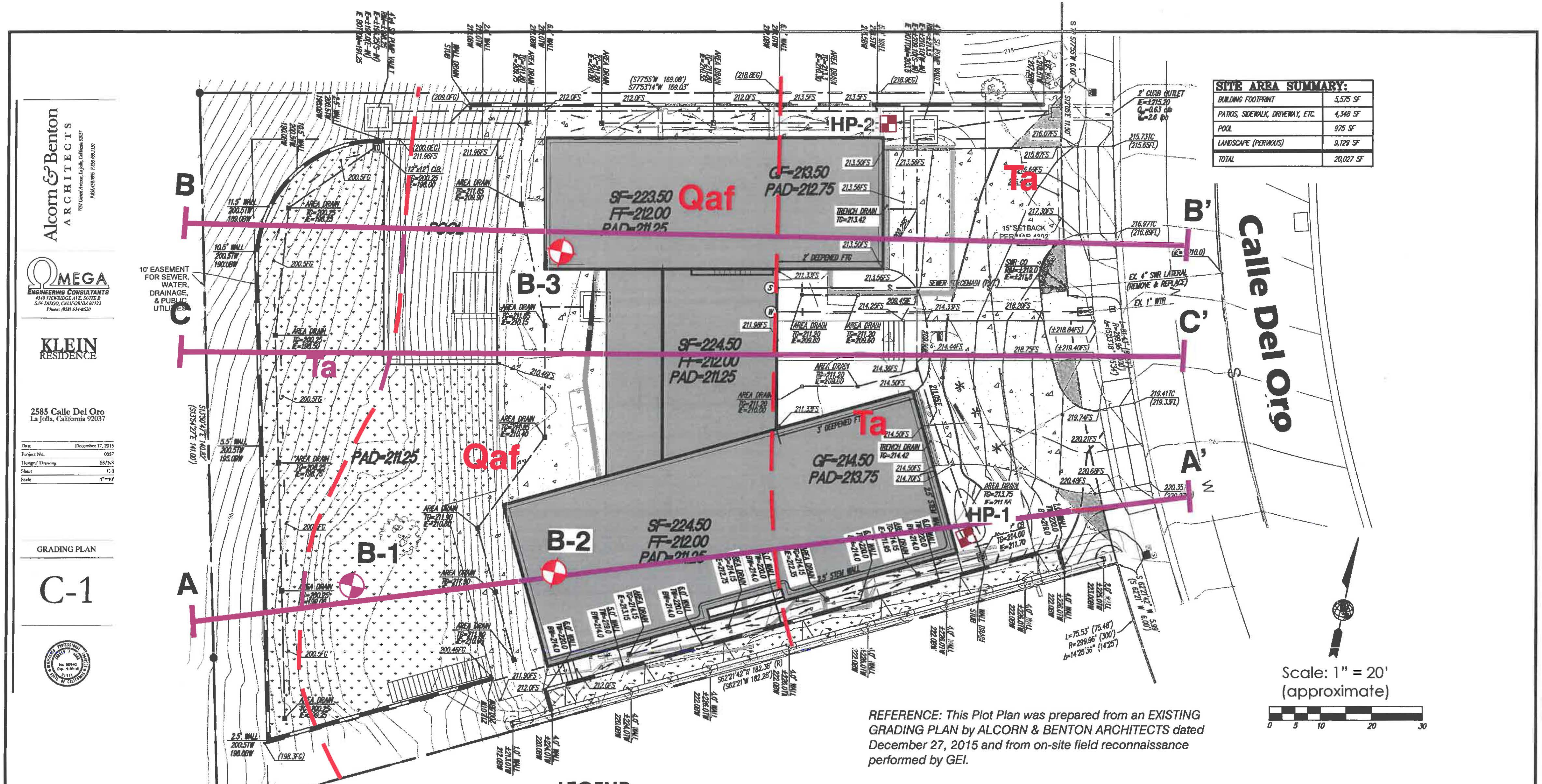
VICINITY MAP



Thomas Bros Guide San Diego County pg 1227-H6

Proposed Klein Residence
2585 Calle Del Oro
La Jolla, CA.

Figure No. I
Job No. 13-10407



REFERENCE: This Plot Plan was prepared from an EXISTING GRADING PLAN by ALCORN & BENTON ARCHITECTS dated December 27, 2015 and from on-site field reconnaissance performed by GEI.

GEOLOGIC LEGEND

Qaf	Artificial Fill
Ta	Ardath Shale Formation
— —	Approximate Geologic Contact

LEGEND

B-3	Approximate Location of Exploratory Boring
HP-3	Approximate Location of Exploratory Handpit
A — A'	Approximate Line of Cross Section

PLOT PLAN

Proposed Klein Residence
2585 Calle Del Oro
La Jolla, CA.
Figure No. II
Job No. 13-10407

Geotechnical Exploration, Inc.

July 2016

13-10407-P3.ai

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Hand Tools	**3' X 3' X 3' Handpit**	**7-18-16**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 220' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–1–2			**CLAYEY SAND**, with abundant roots and rock fragments. Loose to medium dense. Damp. Gray-brown. **FILL (Qaf)**	SC								
2–3			**SANDY CLAY**, with some gypsum crystals. Stiff to hard. Damp. Light gray and yellow-brown. **ARDATH SHALE FORMATION (Ta)**	CL								
3–4			Bottom @ 3'									

▼ PERCHED WATER TABLE

☒ BULK BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S NUCLEAR FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME
Klein Residence

SITE LOCATION
2585 Calle del Oro, La Jolla, CA

JOB NUMBER
13-10407

FIGURE NUMBER
IIId

REVIEWED BY **LDR/JAC**

Geotechnical Exploration, Inc.

LOG No.
HP-1

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Hand Tools	**3' X 3' X 3' Handpit**	**7-18-16**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 220' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			CLAYEY SAND, with some roots and rock fragments. Loose to medium dense. Damp. Gray-brown. **FILL (Qaf)**	SC									
1	▨		**SANDY CLAY**, with some gypsum crystals; moderately fractured. Very stiff to hard. Damp. Yellow-brown. **ARDATH SHALE FORMATION (Ta)**	CL									
2	▨	☒	-- 85% passing #200 sieve.				15.5	112.6			48		
	▨	☒	Bedding: N60°W, 3°NE.										
3			Bottom @ 3'										
4													

- ▼ PERCHED WATER TABLE
- ☒ BULK BAG SAMPLE
- 1 IN-PLACE SAMPLE
- ■ MODIFIED CALIFORNIA SAMPLE
- S NUCLEAR FIELD DENSITY TEST
- ▨ STANDARD PENETRATION TEST

JOB NAME
Klein Residence

SITE LOCATION
2585 Calle del Oro, La Jolla, CA

JOB NUMBER
13-10407

FIGURE NUMBER
IIIe

REVIEWED BY **LDR/JAC**

Geotechnical Exploration, Inc.

LOG No.
HP-2

UPDATE REPORT OF LIMITED GEOTECHNICAL INVESTIGATION

Klein Residence
2585 Calle del Oro
La Jolla, California

JOB NO. 13-10407
21 September 2016

Prepared for:

Trevor and Staci Klein

Geotechnical Exploration, Inc.

SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

21 September 2016

Trevor and Staci Klein
2585 Calle del Oro
La Jolla, CA 92037

Job No. 13-10407

Subject: **Update Report of Limited Geotechnical Investigation**
Klein Residence
2585 Calle del Oro
La Jolla, California

Dear Mr. and Mrs. Klein:

In accordance with your request, ***Geotechnical Exploration, Inc.*** has prepared an updated geotechnical investigation report for the subject site. The original field work, including three exploratory borings, was performed on November 1, 2013, and presented in our "*Report of Limited Geotechnical Investigation*," dated February 10, 2014.

If the conclusions and recommendations presented in this report are incorporated into the design and construction of the proposed residential remodel/additions and associated improvements, it is our opinion that the site is suitable for the proposed project.

This opportunity to be of service is sincerely appreciated. Should you have any questions concerning the following report, please do not hesitate to contact us. Reference to our **Job No. 13-10407** will expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Jaime A. Cerros, P.E.
R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

Jonathan A. Browning
P.G. 9012/C.E.G. 2615
Senior Project Geologist

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REPORT OF LIMITED GEOTECHNICAL INVESTIGATION

Klein Residence
2585 Calle del Oro
La Jolla, California

Job No. 13-10407

The following report presents the findings and recommendations of ***Geotechnical Exploration, Inc.*** for the subject project.

I. PROJECT SUMMARY

It is our understanding, based on discussions with Mr. Paul Benton of Alcorn Benton Architects, that the existing single-family residential structure will be removed and the site will be developed with a new two story, single-family residence over a partial basement. The proposed new site development will also include a level pad on the lower level of the rear yard utilizing Keystone retaining walls with a swimming pool and associated improvements. The proposed new structure will be of standard building materials utilizing conventional foundations with slab on grade floors. The new swimming pool should be supported by caissons.

II. SCOPE OF WORK

The scope of work performed for this investigation included a review of available published information pertaining to the site geology, a site reconnaissance and subsurface exploration program, laboratory testing, geotechnical engineering analysis of the research, field and laboratory data, infiltration testing, and the preparation of this report. The data obtained and the analyses performed were for the purpose of providing design and construction criteria for the project earthwork, new foundations, and slab on-grade floors.

III. SUMMARY OF GEOTECHNICAL & GEOLOGIC FINDINGS

Our subsurface investigation revealed that the site is underlain at depth by dense/hard, adequate bearing sandy silt of the Ardath Shale Formation (Ta), overlain with approximately 1 to 10 feet of silty and clayey fill soil materials primarily on the western portion of the lot. The fill soils are of variable density and will not provide a stable soil base for new additions or associated improvements. As such, it is recommended that new foundations be founded into the underlying formational soils utilizing deepened foundation systems or the existing shallow fill soils at the locations of the proposed foundations be removed and recompacted or new foundations be designed for the existing fill conditions. The planned swimming pool will be located at the top of the existing rear yard slope and as such may require deepened foundations.

The on-site soils should provide adequate bearing strength for new slab on-grade exterior improvements, after proper removal and recompaction of the existing shallow surface soils. As such, we recommend that the existing shallow fill soils be removed and recompacted as part of site preparation prior to placement of slab on-grade exterior improvements in areas where fill soils will not be completely removed by the proposed grade elevation requirements.

In our opinion, the site is suited for the proposed residential construction provided the following recommendations are implemented during site development. Conventional construction techniques and materials can be utilized. Detailed construction plans have not been provided to us for the preparation of this report, however, when completed they should be made available for our review for new or modified recommendations. A shoring wall will most likely be required along the south property line.

In our opinion, there are no geologic hazards on or near the site that would prohibit the construction of the new residential improvements. In addition, the proposed work will not, in our opinion, destabilize or result in settlement of adjacent property if the recommendations presented in this report are implemented.

IV. SITE DESCRIPTION

The approximately 0.4-acre site is more particularly referred to as Lot 13 of Prestwick Estates Unit 1, according to Recorded Map 4392, in the La Jolla area of the City and County of San Diego, State of California. For the location of the site, refer to the Vicinity Map, Figure No. I.

The existing single-story, single-family residence and attached two-car garage are located on a relatively level building pad. The home is constructed of wood frame and stucco construction with slab-on-grade and continuous perimeter footings. A swimming pool and associated improvements are located in the front courtyard area located on the east side of the lot.

Access to the property is provided by a concrete driveway from Calle del Oro Drive at the northeast corner of the lot. Exterior improvements include a swimming pool, concrete flatwork, privacy walls and associated improvements. Vegetation includes ornamental landscaping including decorative shrubs and mature trees.

The rectangular-shaped property is bounded to the north by a similar residential property approximately 8 feet lower in elevation; to the south by a similar residential property approximately 8 feet higher in elevation; to the east by Calle del Oro; and to the west by a westerly descending, approximately 12- to 18-foot-high composite fill/cut slope that abuts an existing residential property at its downslope terminus.

The site consists of a relatively level building pad at an approximate elevation of 219 feet above Mean Sea Level (AMSL). Elevations across the property range from approximately 225 feet AMSL at the southeast corner of the property, down to 180 feet AMSL at the northwest corner of the property. Information concerning approximate elevations across the site was obtained from an undated topographic survey map by Pasco Laret Suiter & Associates.

V. FIELD INVESTIGATION

A. *Exploratory Excavations*

Three exploratory borings were placed on the western portion of the site in areas near where the originally proposed additions and improvements were to be located on November 1, 2013, and where access and soil conditions allowed. In addition, two exploratory handpits were excavated on the eastern portion of the site where the current residential structure is proposed and where soil conditions allowed (for exploratory boring and handpit locations, refer to Figure No. II). The borings and handpits were excavated to depths ranging from 3 to 11½ feet in order to obtain representative soil samples and to define a soil profile across the lot.

The soils encountered in the exploratory borings and handpits were observed and logged by our field representative and samples were taken of the predominant soils. Excavation logs have been prepared on the basis of our observations and laboratory testing. The results have been summarized on Figure Nos. III and IV.

The soils encountered in the excavations have been classified in general conformance with the Unified Soil Classification System (refer to Appendix A).

B. Slope Observations

The stability of the existing slopes should not be affected by the planned residential construction if proper drainage conditions are implemented and maintained. The existing composite cut/fill slope is at an approximate gradient of 1.5:1.0 (horizontal to vertical). The slope was observed to be in generally good condition with no evidence of instability or prior slope failure.

We have been asked to address the level of pre-existing disturbance of the hillside property related to the City of San Diego requirements for steep slopes and Environmentally Sensitive Land (ESL). It should be noted that the descending rear slope is considered manufactured. Fill was placed on the upper slope and the lower portion of the slope was cut during the original mass grading and site development in the late 1960s.

C. Infiltration Testing

We performed simple open pit testing at one location at a depth of 36 inches per the requirements of the City of San Diego's Storm Water Standards, BMP Design Manual, in accordance with Appendix D. The infiltration basin has been proposed in the northeast corner of the property, and is the most feasible location on the property (i.e., gentle gradient, away from structures, away from utilities and retaining walls, and away from existing slopes). Testing at infiltration test location (INF-1), revealed simple open pit test rate of 20 minutes/inch (refer to Appendix D for simple open pit test rates and converted infiltration rates). The simple open pit test rate results have been converted to infiltration rates, using the Porchet Method and indicate infiltration rate of 1.636 inches/hour. Based on the results of our simple open pit testing and review of USDA soil maps, the site has been assigned to soil group Hydrologic Soil Group (HSG) D. As part of our geologic/geotechnical site evaluation we considered the following issues:

1. The site is ***not*** subject to high groundwater conditions (within 10 feet of the base of the infiltration facility).

2. The site is ***not*** in close proximity to a known contaminated soil site.

3. The site ***is*** underlain by hard formational sandy clay soils, but ***not*** subject to hydroconsolidation.

4. The site ***has*** infiltration rate of 1.636-inches/hour.

5. The site ***does have*** a silt plus clay percentage of greater than 50.

6. The site ***is*** underlain at relatively shallow depths by practically impermeable formational soils.

7. The site is ***not*** located within 100 feet from a drinking water well.

8. The site is ***not*** located within 100 feet from an on-site septic system or designated expansion area.

9. The site ***is*** located adjacent to a slope steeper than 25 percent.

Based on the results of our simple open pit falling head testing and evaluation of the infiltration rate, it is our professional opinion that the proposed bio-retention basin has appreciable infiltration rates for the design of partial infiltration BMPs. Based on items 3, 5, 6 and 9 listed above, however, they do not have favorable soil conditions. As such, we recommend all bio-retention facilities be lined with impermeable liner and drained.

VI. FIELD & LABORATORY TESTS AND SOIL INFORMATION

A. Field Tests

Relatively undisturbed samples were obtained from the borings by driving a 3-inch outside-diameter (O.D.) by 2-3/8-inch inside-diameter (I.D.) split-tube sampler a distance of 12 inches. Standard Penetration Tests were also performed by using a 140-pound weight falling 30 inches to drive a 2-inch O.D. by 1⅜-inch I.D. sampler tube a distance of 18 inches. The number of blows required to drive the sampler the last 12 inches was recorded for use in evaluation of the soil consistency. The following chart provides an in-house correlation between the number of blows and the consistency of the soil for the Standard Penetration Test and the 3-inch sampler. Blow counts are provided on Figure Nos. IIIa-c.

Soil	***Density Designation***	***2-inch O.D. Sampler Blows/Foot***	***3-inch O.D. Sampler Blows/Foot***
Sand and Non-plastic Silt	Very loose	0-4	0-7
	Loose	5-10	8-20
	Medium	11-30	21-53
	Dense	31-50	54-98
	Very Dense	Over 50	Over 98

Soil	**Density Designation**	**2-inch O.D. Sampler Blows/Foot**	**3-inch O.D. Sampler Blows/Foot**
Clay and Plastic Silt	Very soft	0-2	0-2
	Soft	3-4	3-4
	Firm	5-8	5-9
	Stiff	9-15	10-18
	Very Stiff	15-30	19-45
	Hard	31-60	46-90
	Very Hard	Over 60	Over 90

In general the tests performed in the field included: the Standard Practice for Soil Investigation and Sampling by Auger Borings (ASTM D1452), Test Method for Penetration Test and Split-barrel Sampling of Soils (ASTM D1586) and Standard Practice for Ring-lined Barrel Sampling of Soils (ASTM D3550). Bulk (disturbed) samples of the encountered soils were also retrieved for subsequent laboratory testing.

Blow counts in the formational soils ranged from 40 to 56 per foot (dense/hard) for the 2-inch-diameter (SPT) sampler. Blow counts in the fill soils yielded 8 to 60 per foot (loose to very dense) with the 3-inch-diameter sampler and 6 to 20 per foot (loose to medium dense) for the 2-inch-diameter sampler.

B. <u>Laboratory Tests</u>

Laboratory tests were performed on retrieved soil samples in order to evaluate their physical and mechanical properties and their ability to support the proposed residential additions and improvements. Test results are presented on Figure Nos. III and IV. The following tests were conducted on representative soil samples:

1. *Moisture Content (ASTM D2216-10)*
2. *Density Measurements (ASTM D2937-10)*
3. *Laboratory Compaction Characteristics (ASTM D1557-12)*
4. *Determination of Percentage of Particles Smaller than #200 Sieve (ASTM D1140-06)*
5. *Standard Test Method for Expansion Index of Soils (ASTM D4829-11)*

The moisture content of a soil sample (*ASTM D2216*) is a measure of the water content, expressed as a percentage of the dry weight of the sample. Moisture content and density measurements (*ASTM D2937*) were performed to establish the in situ moisture and density of samples retrieved from the exploratory borings. The dry soil weight was compared to the laboratory maximum dry density of the same soil to determine relative compaction.

Laboratory compaction values (*ASTM D1557*) establish the optimum moisture content and the laboratory maximum dry density of the tested soils. The relationship between the moisture and density of remolded soil samples helps to establish the relative compaction of the existing fill soils and soil compaction conditions to be anticipated during any future grading operation.

The passing −200 sieve size analysis (*ASTM D1140*) aids in classification of the tested soils based on their fine material content and provides qualitative information related to engineering characteristics such as expansion potential, permeability, and shear strength.

The expansion potential of soils is determined, when necessary, utilizing the Standard Test Method for Expansion Index of Soils (*ASTM D4829*). In accordance with the Standard (Table 5.3), potentially expansive soils are classified as follows:

EXPANSION INDEX	*POTENTIAL EXPANSION*
0 to 20	Very low
21 to 50	Low
51 to 90	Medium
91 to 130	High
Above 130	Very high

Based on the test results, the existing clayey fill soils have a medium expansion potential, with a maximum measured expansion index of 68. Based on our experience with similar soils, it is our opinion that the on-site formational soils also possess a medium expansion potential.

Based on the laboratory test data, our observations of the primary soil types, and our previous experience with laboratory testing of similar soils, our Geotechnical Engineer has assigned values for friction angle, coefficient of friction, and cohesion for those soils that will have significant lateral support or load bearing functions on the project. These values have been utilized in determining the recommended bearing value as well as active and passive earth pressure design criteria.

VII. REGIONAL GEOLOGIC DESCRIPTION

San Diego County has been divided into three major geomorphic provinces: the Coastal Plain, the Peninsular Ranges and the Salton Trough. The Coastal Plain exists west of the Peninsular Ranges. The Salton Trough is east of the Peninsular Ranges. These divisions are the result of the basic geologic distinctions between the areas. Mesozoic metavolcanic, metasedimentary and plutonic rocks predominate in the Peninsular Ranges with primarily Cenozoic sedimentary rocks to the west and east of this central mountain range (Demere, 1997).

In the Coastal Plain region, where the subject property is located, the "*basement*" consists of Mesozoic crystalline rocks. Basement rocks are also exposed as high relief areas (e.g., Black Mountain northeast of the subject property and Cowles Mountain near the San Carlos area of San Diego). Younger Cretaceous and Tertiary sediments lap up against these older features. The Cretaceous sediments form the local basement rocks on the Point Loma area. These sediments form a "layer cake" sequence of marine and non-marine sedimentary rock units, with some formations up to 140 million years old. Faulting related to the La Nacion and Rose Canyon Fault zones has broken up this sequence into a number of distinct fault blocks in the southwestern part of the county. Northwestern portions of the county are relatively undeformed by faulting (Demere, 1997).

The Peninsular Ranges form the granitic spine of San Diego County. These rocks are primarily plutonic, forming at depth beneath the earth's crust 140 to 90 million years ago as the result of the subduction of an oceanic crustal plate beneath the North American continent. These rocks formed the much larger Southern California batholith. Metamorphism associated with the intrusion of these great granitic masses affected the much older sediments that existed near the surface over that period of time. These metasedimentary rocks remain as roof pendants of marble, schist, slate, quartzite and gneiss throughout the Peninsular Ranges. Locally, Miocene-age volcanic rocks and flows have also accumulated within these mountains (e.g., Jacumba Valley). Regional tectonic forces and erosion over time have uplifted and unroofed these granitic rocks to expose them at the surface (Demere, 1997).

The Salton Trough is the northerly extension of the Gulf of California. This zone is undergoing active deformation related to faulting along the Elsinore and San Jacinto Fault Zones, which are part of the major regional tectonic feature in the southwestern portion of California, the San Andreas Fault Zone. Translational movement along these fault zones has resulted in crustal rifting and subsidence. The Salton Trough, also referred to as the Colorado Desert, has been filled with sediments to depth of approximately 5 miles since the movement began in the early Miocene, 24 million years ago. The source of these sediments has been the local mountains as well as the ancestral and modern Colorado River (Demere, 1997).

As indicated previously, the San Diego area is part of a seismically active region of California. It is on the eastern boundary of the Southern California Continental Borderland, part of the Peninsular Ranges Geomorphic Province. This region is part of a broad tectonic boundary between the North American and Pacific Plates. The actual plate boundary is characterized by a complex system of active, major, right-lateral strike-slip faults, trending northwest/southeast. This fault system extends eastward to the San Andreas Fault (approximately 70 miles from San Diego) and westward to the San Clemente Fault (approximately 50 miles off-shore from San Diego) (Berger and Schug, 1991).

In California, major earthquakes can generally be correlated with movement on active faults. As defined by the California Division of Mines and Geology (Hart, E.W., 1980), an "active" fault is one that has had ground surface displacement within Holocene time (about the last 11,000 years). Additionally, faults along which major historical earthquakes have occurred (about the last 210 years in California) are also considered to be active (Association of Engineering Geologist, 1973). The California Division of Mines and Geology (now the California Geological Survey) defines a *"potentially active"* fault as one that has had ground surface displacement during Quaternary time, that is, between 11,000 and 1.6 million years (Hart, E.W., 1980).

During recent history, prior to April 2010, the San Diego County area was relatively quiet seismically. No fault ruptures or major earthquakes had been experienced in historic time within the greater San Diego area. Since earthquakes have been recorded by instruments (since the 1930s), the San Diego area had experienced scattered seismic events with Richter magnitudes (M) generally less than M4.0. During June 1985, a series of small earthquakes occurred beneath San Diego Bay, three of which had recorded magnitudes of M4.0 to M4.2. In addition, the Oceanside earthquake of July 13, 1986, located approximately 26 miles offshore of the City of Oceanside, had a magnitude of M5.3 (Hauksson and Jones, 1988).

On June 15, 2004, a M5.3 earthquake occurred approximately 45 miles southwest of downtown San Diego (26 miles west of Rosarito, Mexico). Although this earthquake was widely felt, no significant damage was reported. Another widely felt earthquake on a distant southern California fault was a M5.4 event that took place on July 29, 2008, west-southwest of the Chino Hills area of Riverside County.

Several earthquakes ranging from M5.0 to M6.0 occurred in northern Baja California, centered in the Gulf of California on August 3, 2009. These were felt in San Diego but no injuries or damage was reported. A M5.8 earthquake followed by a M4.9 aftershock occurred on December 30, 2009, centered about 20 miles south of the Mexican border city of Mexicali. These were also felt in San Diego, swaying high-rise buildings, but again no significant damage or injuries were reported.

On Easter Sunday April 4, 2010, a large earthquake occurred in Baja California, Mexico. It was widely felt throughout the U.S. southwest including Phoenix, Arizona and San Diego in California. It significantly affected Mexicali, Mexico. This M7.2 event, the Sierra El Mayor earthquake, occurred in northern Baja California, approximately 40 miles south of the Mexico-USA border at relatively shallow depth along the principal plate boundary between the North American and Pacific plates.

According to the U. S. Geological Survey, this is an area with a high level of historical seismicity, and it has recently also been seismically active, though this is the largest event to strike in this area since 1892. The April 4, 2010, earthquake appears to have been larger than the M6.9 earthquake in 1940 or any of the early 20th century events (e.g., 1915 and 1934) in this region of northern Baja California. The event caused widespread damage to structures, closure of businesses, government offices and schools, power outages, displacement of people from their homes and injuries in the nearby major metropolitan areas of Mexicali in Mexico and Calexico in Southern California. Estimates of the cost of the damage range to over $100 million.

This event's aftershock zone extends significantly to the northwest, overlapping with the portion of the fault system that is thought to have ruptured in 1892. Some structures in the San Diego area experienced minor damage and there were some injuries. Ground motions for the April 4, 2010, main event, recorded at stations in San Diego and reported by the California Strong Motion Instrumentation Program (CSMIP), ranged up to 0.058g. Aftershocks from this event continue to the date of this report along the trend northwest and south of the original event, including within San Diego County, closer to the San Diego metropolitan area. There have been hundreds of these earthquakes including events up to M5.7.

On July 7, 2010, a M5.4 earthquake occurred in Southern California at 4:53 pm (Pacific Time) about 30 miles south of Palm Springs, 25 miles southwest of Indio, and 13 miles north-northwest of Borrego Springs. The earthquake occurred near the Coyote Creek segment of the San Jacinto Fault. The earthquake exhibited right lateral slip consistent with the direction of movement on the San Jacinto Fault. The earthquake was felt throughout Southern California, with strong shaking near the epicenter. It was followed by more than 60 aftershocks of M1.3 and greater during the first hour. Seismologists expect continued aftershock activity.

In the last 50 years, there have been four other earthquakes in the magnitude M5.0 range within 20 kilometers of the Coyote Creek segment: M5.8 in 1968, M5.3 on 2/25/1980, M5.0 on 10/31/2001, and M5.2 on 6/12/2005. The biggest earthquake near this location was the M6.0 Buck Ridge earthquake on 3/25/1937.

VIII. SITE-SPECIFIC SOIL & GEOLOGIC DESCRIPTION

A. Stratigraphy

Our field work, reconnaissance and review of the *"Geologic Map of the La Jolla Quadrangle"* contained within California Division of Mines and Geology (now the California Geological Survey) Bulletin 200 *"Geology of the San Diego Metropolitan Area, California"* (Michael P. Kennedy, 1975) and the updated geologic maps by Kennedy and Tan, 2005 and 2008, *"Geologic Map of San Diego, 30'x60' Quadrangle, CA,"* indicate that the site is underlain by Eocene-age Ardath Shale (Ta) formational materials. The formational soils are overlain by approximately 1 to 10 feet of fill soil on the building pad (refer to the excavation logs, Figure Nos. IIIa-c). Figure No. V presents a plan view geologic map (Kennedy and Tan, 2008) of the general area of the site. A geologic cross section through the planned project area and slope is presented as Figure No. VI. Figure No. VII displays the geologic hazards of the area.

Fill Soils (Qaf): The lot is overlain by approximately 1 to 10 feet of fill soils that thicken across the lot in a westerly direction. The fill soils were encountered at all boring and handpit locations and consist of gray-brown silty sand and light gray to light brown and orange silty clay with some caliche, sand and siltstone fragments. The encountered fill soils were generally in a loose to medium dense, dry to moist condition and are considered to have a medium expansion potential. Refer to Figure Nos. III and IV for details.

Ardath Shale Formation (Ta): Formational materials of the Ardath Shale Formation were encountered at all exploratory boring and handpit locations below the fill soils. These formational soils were encountered at depths of 5 feet, 8 feet and 10 feet, respectively, at the locations of borings B-1, B-2, and B-3, and from 1 to 2 feet at the exploratory handpit locations HP-1 and HP-2.

The formational soils consist of dense/ hard, dark gray, sandy siltstone with some clay and are considered to have a medium expansion potential. The formational soils have good bearing strength characteristics (refer to Figure Nos. IIIa-e).

B. ***Structure***

Based on our observations, the site is underlain by relatively stable formational materials and no adverse geologic conditions are expected. Exploratory handpit HP-2, exposed bedding attitude of north 60 degrees west, dipping 3 degrees to the northeast. Mapping by Kennedy and Tan, 2008, indicates bedding attitudes within the Ardath Shale Formation in the vicinity of the subject site strike approximately north 30 to 40 degrees to the northwest and dip 3 to 5 degrees to the northeast. These dips are into the hillside (or parallel to the hillside) and, therefore, are considered to be a relatively stable geologic condition.

A review of the City of San Diego Geologic Hazards Map indicates that no faults are mapped on the site. The active Rose Canyon Fault Zone (RCFZ) is mapped approximately 1/2 mile west of the property.

IX. GEOLOGIC HAZARDS

A review of the City of San Diego Geologic Hazards Map Sheet No. 29 indicates that the site is located in a moderate risk geologic hazard area designated as Category 26. Category 26 is identified as being underlain by *"slide-prone formations"* specifically the Ardath Formation with *"unfavorable geologic structure."* In our opinion, the *"unfavorable geologic structure"* description does not apply due to the favorable dips within the formational materials. An excerpted portion of the Geologic Hazards Map Sheet 29 and the legend are presented as Figure No. VII.

The following is a discussion of the geologic conditions and hazards common to this area of the City of San Diego, as well as project-specific geologic information relating to development of the subject property.

A. Local and Regional Faults

Reference to the geologic map of the area, Figure No. V (Kennedy and Tan, 2008), and the City of San Diego Seismic Safety Study, Geologic Hazards Map No. 29, Figure No. VII, indicates that no faults are mapped on the site. In our explicit professional opinion, neither an active fault nor a potentially active fault underlies the site.

Rose Canyon Fault: The Rose Canyon Fault Zone (Mount Soledad and Rose Canyon Faults) is mapped 0.5 mile southwest of the subject site. The Rose Canyon Fault is mapped trending north-south from Oceanside to downtown San Diego, from where it appears to head southward into San Diego Bay, through Coronado and offshore. The Rose Canyon Fault Zone is considered to be a complex zone of onshore and offshore, en echelon strike slip, oblique reverse, and oblique normal faults. The Rose Canyon Fault is considered to be capable of generating an M7.2 earthquake and is considered microseismically active, although no significant recent earthquakes are known to have occurred on the fault.

Investigative work on faults that are part of the Rose Canyon Fault Zone at the Police Administration and Technical Center in downtown San Diego, at the SDG&E facility in Rose Canyon, and within San Diego Bay and elsewhere within downtown San Diego, has encountered offsets in Holocene (geologically recent) sediments. These findings confirm Holocene displacement on the Rose Canyon Fault, which was designated an "*active*" fault in November 1991 (Hart E.W. and W.A. Bryant, 2007, Fault-Rupture Hazard Zones in California, California Geological Survey Special Publication 42).

In a report compiled by Rockwell et al. (2012) for Southern California Edison, it is suggested that the recurrence interval for earthquakes on the RCFZ is in the range of 400 to 500 years, with the most recent earthquake (MRE) nearly 500 years ago. The report indicates the slip rate on the RCFZ is not well constrained but a compilation of the latest research implies a long-term slip rate of approximately 2 mm/year.

Coronado Bank Fault: The Coronado Bank Fault is located approximately 11 miles southwest of the site. Evidence for this fault is based upon geophysical data (acoustic profiles) and the general alignment of epicenters of recorded seismic activity (Greene, 1979). The Oceanside earthquake of M5.3 recorded July 13, 1986, is known to have been centered on the fault or within the Coronado Bank Fault Zone. Although this fault is considered active, due to the seismicity within the fault zone, it is significantly less active seismically than the Elsinore Fault (Hileman, 1973). It is postulated that the Coronado Bank Fault is capable of generating a M7.6 earthquake and is of great interest due to its close proximity to the greater San Diego metropolitan area.

Newport-Inglewood Fault: The Newport-Inglewood Fault Zone is located approximately 23 miles northwest of the site. A significant earthquake (M6.4) occurred along this fault on March 10, 1933. Since then no additional significant events have occurred. The fault is believed to have a slip rate of approximately 0.6 mm/yr with an unknown recurrence interval. This fault is believed capable of producing an earthquake of M6.0 to M7.4 (SCEC, 2004).

Elsinore Fault: The Elsinore Fault is located approximately 36 to 54 miles east and northeast of the site. The fault extends approximately 200 km (125 miles) from the Mexican border to the northern end of the Santa Ana Mountains. The Elsinore Fault zone is a 1- to 4-mile-wide, northwest-southeast-trending zone of discontinuous and en echelon faults extending through portions of Orange, Riverside, San Diego, and Imperial Counties. Individual faults within the Elsinore Fault Zone range from less than 1 mile to 16 miles in length. The trend, length and geomorphic expression of the Elsinore Fault Zone identify it as being a part of the highly active San Andreas Fault system.

Like the other faults in the San Andreas system, the Elsinore Fault is a transverse fault showing predominantly right-lateral movement. According to Hart, et al. (1979), this movement averages less than 1 centimeter per year. Along most of its length, the Elsinore Fault Zone is marked by a bold topographic expression consisting of linearly aligned ridges, swales and hallows. Faulted Holocene alluvial deposits (believed to be less than 11,000 years old) found along several segments of the fault zone suggest that at least part of the zone is currently active.

Although the Elsinore Fault Zone belongs to the San Andreas set of active, northwest-trending, right-slip faults in the southern California area (Crowell, 1962), it has not been the site of a major earthquake in historic time, other than a M6.0 earthquake near the town of Elsinore in 1910 (Richter, 1958; Toppozada and Parke, 1982). However, based on length and evidence of late-Pleistocene or Holocene displacement, Greensfelder (1974) has estimated that the Elsinore Fault Zone is reasonably capable of generating an earthquake with a magnitude as large as M7.5. Study and logging of exposures in trenches placed in Glen Ivy Marsh across the Glen Ivy North Fault (a strand of the Elsinore Fault Zone between Corona and Lake Elsinore), suggest a maximum earthquake recurrence interval of 300 years, and when combined with previous estimates of the long-term horizontal slip rate of 0.8 to 7.0 mm/year, suggest typical earthquake magnitudes of M6.0 to M7.0 (Rockwell, 1985). More recently, the California Geologic Survey (2002) considers the Elsinore Fault capable of producing an earthquake of M6.8 to M7.1.

San Jacinto Fault: The San Jacinto Fault is located 59 to 65 miles to the northeast of the site. The San Jacinto Fault Zone consists of a series of closely spaced faults, including the Coyote Creek Fault, that form the western margin of the San Jacinto Mountains. The fault zone extends from its junction with the San Andreas Fault in San Bernardino, southeasterly toward the Brawley area, where it continues south of the international border as the Imperial Transform Fault (Earth Consultants International, 2009).

The San Jacinto Fault zone has a high level of historical seismic activity, with at least 10 damaging earthquakes (M6.0 to M7.0) having occurred on this fault zone between 1890 and 1986. Earthquakes on the San Jacinto Fault in 1899 and 1918 caused fatalities in the Riverside County area. Offset across this fault is predominantly right-lateral, similar to the San Andreas Fault, although some investigators have suggested that dip-slip motion contributes up to 10% of the net slip (ECI, 2009).

The segments of the San Jacinto Fault that are of most concern to major metropolitan areas are the San Bernardino, San Jacinto Valley and Anza segments. Fault slip rates on the various segments of the San Jacinto are less well constrained than for the San Andreas Fault, but the available data suggest slip rates of 12 ±6 mm/yr for the northern segments of the fault, and slip rates of 4 ±2 mm/yr for the southern segments. For large ground-rupturing earthquakes on the San Jacinto fault, various investigators have suggested a recurrence interval of 150 to 300 years. The Working Group on California Earthquake Probabilities (WGCEP, 2008) has estimated that there is a 31 percent probability that an earthquake of M6.7 or greater will occur within 30 years on this fault. Maximum credible earthquakes of M6.7, M6.9 and M7.2 are expected on the San Bernardino, San Jacinto Valley and Anza segments, respectively, capable of generating peak horizontal ground accelerations of 0.48 to 0.53 g in the County of Riverside, (ECI, 2009). A M5.4 earthquake occurred on the San Jacinto Fault on July 7, 2010.

The United States Geological Survey has issued the following statements with respect to the recent seismic activity on southern California faults:

The San Jacinto fault, along with the Elsinore, San Andreas, and other faults, is part of the plate boundary that accommodates about 2 inches/year of motion as the Pacific plate moves northwest relative to the North American plate. The largest recent earthquake on the San Jacinto fault, near this location, the M6.5 1968 Borrego Mountain earthquake April 8, 1968, occurred about 25 miles southeast of the July 7, 2010, M5.4 earthquake.

This M5.4 earthquake follows the 4th of April 2010, Easter Sunday, Mw7.2 earthquake, located about 125 miles to the south, well south of the US Mexico international border. A M4.9 earthquake occurred in the same area on June 12th at 8:08 pm (Pacific Time). Thus this section of the San Jacinto fault remains active.

Seismologists are watching two major earthquake faults in southern California. The San Jacinto fault, the most active earthquake fault in southern California, extends for more than 100 miles from the international border into San Bernardino and Riverside, a major metropolitan area often called the Inland Empire. The Elsinore fault is more than 110 miles long, and extends into the Orange County and Los Angeles area as the Whittier fault. The Elsinore fault is capable of a major earthquake that would significantly affect the large metropolitan areas of southern California. The Elsinore fault has not hosted a major earthquake in more than 100 years. The occurrence of these earthquakes along the San Jacinto fault and continued aftershocks demonstrates that the earthquake activity in the region remains at an elevated level. The San Jacinto fault is known as the most active earthquake fault in southern California. Caltech and USGS seismologist continue to monitor the ongoing earthquake activity using the Caltech/USGS Southern California Seismic Network and a GPS network of more than 100 stations.

B. ***Other Geologic Hazards***

Ground Rupture: Ground rupture is characterized by bedrock slippage along an established fault and may result in displacement of the ground surface. For ground rupture to occur along a fault, an earthquake usually exceeds M5.0. If a M5.0 earthquake were to take place on a local fault, an estimated surface-rupture length

1 mile long could be expected (Greensfelder, 1974). Our investigation indicates that the subject site is not directly on a known active fault trace and, therefore, the risk of ground rupture is remote.

Ground Shaking: Structural damage caused by seismically induced ground shaking is a detrimental effect directly related to faulting and earthquake activity. Ground shaking is considered to be the greatest seismic hazard in San Diego County. The intensity of ground shaking is dependent on the magnitude of the earthquake, the distance from the earthquake, and the seismic response characteristics of underlying soils and geologic units. Earthquakes of M5.0 or greater are generally associated with notable to significant damage. It is our opinion that the most serious damage to the site would be caused by a large earthquake originating on a nearby strand of the Rose Canyon Fault Zone. Although the chance of such an event is remote, it could occur within the useful life of the structure.

Landslides: Based upon our geotechnical investigation, review of the geologic maps (Kennedy and Tan, 2008 and Kennedy, 1975), review of the referenced City of San Diego Seismic Safety Study -- Geologic Hazards Map Sheet 29 and stereo-pair aerial photographs (3-29-53, AXN-8M-1 and 2), there are no known or suspected ancient landslides located on the site.

Slope Stability: We have performed slope stability analysis based on our exploratory borings, the laboratory test results from retrieved soil samples collected during the drilling, our field review of site conditions, review of aerial photos, review of pertinent documents and geologic maps, and our experience with similar formational units in the La Jolla area of San Diego. We performed slope stability calculations using Taylor's charts and conventional equations for gross and shallow stability as well as the *SLIDE6* program. The gross slope stability analyses were performed along cross sections A-A′, B-B′ and C-C′ (see Figure No. VI). The locations of the cross section is presented on the Plot Plan and Site-Specific Geologic Map, Figure No. II. Based on our slope stability analysis, a factor of safety (FS) less than 1.5 against gross or shallow slope failure does not exist on the property. In our professional opinion, the site will have a factor of safety of 1.5 or greater following the proposed construction. Refer to Appendix C for the results of the analyses.

Liquefaction: The liquefaction of saturated sands during earthquakes can be a major cause of damage to buildings. Liquefaction is the process by which soils are transformed into a viscous fluid that will flow as a liquid when unconfined. It occurs primarily in loose, saturated sands and silts when they are sufficiently shaken by an earthquake.

On this site, the risk of liquefaction of foundation materials due to seismic shaking is considered to be remote due to the dense nature of the natural-ground material, the anticipated high density of the proposed recompacted fill, and the lack of a shallow static groundwater surface under the site. The site does not have a potential for soil strength loss to occur due to a seismic event.

Tsunami: A tsunami is a series of long waves generated in the ocean by a sudden displacement of a large volume of water. Underwater earthquakes, landslides, volcanic eruptions, meteoric impacts, or onshore slope failures can cause this displacement. Tsunami waves can travel at speeds averaging 450 to 600 miles per hour. As a tsunami nears the coastline, its speed diminishes, its wave length decreases, and its height increases greatly. After a major earthquake or other tsunami-inducing activity occurs, a tsunami could reach the shore within a few minutes. One coastal community may experience no damaging waves while another may experience very destructive waves. Some low-lying areas could experience severe inland inundation of water and deposition of debris more than 3,000 feet inland.

Wave heights and run-up elevations from tsunami along the San Diego Coast have historically fallen within the normal range of the tides (Joy 1968). The largest tsunami effect recorded in San Diego since 1950 was May 22, 1960, which had a maximum wave height 2.1 feet (NOAA, 1993). In this event, 80 meters of dock were destroyed and a barge sunk in Quivera Basin. Other tsunamis felt in San Diego County occurred on November 5, 1952, with a wave height of 2.3 feet caused by an earthquake in Kamchatka; March 9, 1957, with a wave height of 1.5 feet; May 22, 1960, at 2.1 feet; March 27, 1964, with a wave height of 3.7 feet and September 29, 2009, with a wave height of 0.5 feet. It should be noted that damage does not necessarily occur in direct relationship to wave height, illustrated by the fact that the damage caused by the 2.1-foot wave height in 1960 was worse than damage caused by several other tsunamis with higher wave heights.

The site is located over 2000 feet from the Pacific Ocean strand line at a pad elevation of over 200 feet. It is unlikely that a tsunami would affect the lot. The site is not mapped within a possible inundation zone on the California Geological Survey's 2009 *"Tsunami Inundation Map for Emergency Planning, La Jolla Quadrangle, San Diego County."*

Geologic Hazards Summary: It is our opinion, based upon a review of the available geologic maps, our research and our site investigation, that the site is underlain by relatively stable formational materials (and shallow fill soils), and is suited for the proposed residential structure and associated improvements provided the recommendations herein are implemented. No significant geologic hazards are known to exist on the site that would prevent the proposed construction. In our explicit professional opinion, no *"active"* or *"potentially active"* faults underlie the project site.

The most significant geologic hazard at the site is anticipated ground shaking from earthquakes on active Southern California and Baja California faults. The United States Geologic Survey has issued statements indicating that seismic activity in Southern California may continue at elevated levels with increased risk to major metropolitan areas near the Elsinore and San Jacinto faults. The San Jacinto fault is too far from the subject property to present a seismic risk. To date, the nearest known "active" faults to the subject site are the northwest-trending Rose Canyon Fault, Coronado Bank Fault and a portion of the Elsinore Fault.

X. GROUNDWATER

Groundwater and/or perched water conditions were not encountered at the explored excavation locations and we do not expect significant groundwater problems to develop in the future ***if proper drainage is maintained on the property***. The potential does exist for perched water conditions to occur if rainwater and irrigation waters are allowed to infiltrate through the upper, more permeable fill soils and encounter less permeable natural ground materials.

It should be kept in mind that construction operations may change surface drainage patterns and/or reduce permeabilities due to the densification of compacted soils. Such changes of surface and subsurface hydrologic conditions, plus irrigation of landscaping or significant increases in rainfall, may result in the appearance of surface or near-surface water at locations where none existed previously. The appearance of such water is expected to be localized and cosmetic in nature, if good positive drainage is implemented, as recommended in this report, during and at the completion of construction.

On properties such as the subject site where dense, low permeability soils exist at shallow depths, even normal landscape irrigation practices on the property or neighboring properties, or periods of extended rainfall, can result in shallow "perched" water conditions. The perching (shallow depth) accumulation of water on a low permeability surface can result in areas of persistent wetting and drowning of lawns, plants and trees. Resolution of such conditions, should they occur, may require site-specific design and construction of subdrain and shallow "wick" drain dewatering systems.

Subsurface drainage with a properly designed and constructed subdrain system will be required behind proposed below-ground building retaining walls. Additional recommendations may be required at the time of construction.

It must be understood that unless discovered during initial site exploration or encountered during site construction operations, it is extremely difficult to predict if or where perched or true groundwater conditions may appear in the future. When site fill or formational soils are fine-grained and of low permeability, water problems may not become apparent for extended periods of time.

Water conditions, where suspected or encountered during construction, should be evaluated and remedied by the project civil and geotechnical consultants. The project developer and property owner, however, must realize that post-construction appearances of groundwater may have to be dealt with on a site-specific basis.

XII. RECOMMENDATIONS

The following recommendations are based upon the practical field investigation conducted by our firm, and resulting laboratory tests, in conjunction with our knowledge and experience with similar soils in the La Jolla area.

The opinions, conclusions, and recommendations presented in this report are contingent upon ***Geotechnical Exploration, Inc.*** being retained to review the final plans and specifications as they are developed and to observe the site earthwork and installation of foundations. Accordingly, we recommend that the following paragraph be included on the grading and foundation plans for the project:

> If the geotechnical consultant of record is changed for the project, the work shall be stopped until the replacement has agreed in writing to accept the responsibility within their area of technical competence for approval upon completion of the work. It shall be the responsibility of the permittee to notify the governing agency in writing of such change prior to the commencement or recommencement of grading and/or foundation installation work.

A. Seismic Design Criteria

1. *Seismic Design Criteria:* The proposed structures should be designed in accordance with the 2013 CBC, which incorporates by reference the ASCE 7-10 for seismic design. We recommend the following parameters be utilized. We have determined the mapped spectral acceleration values for the site based on a latitude of 32.8550 degrees and longitude of -117.2489 degrees, utilizing a program titled "U.S. Seismic Design Maps and Tools," provided by the USGS, which provides a solution for ASCE 7-10 (2013 CBC) utilizing digitized files for the Spectral Acceleration maps.

In addition, we have assigned a Site Classification of D. The response parameters for design are presented in the following table. The design Spectral Acceleration (SA) vs. Period (T) is shown on Appendix B.

TABLE I
Mapped Spectral Acceleration Values and Design Parameters

S_s	S_1	F_a	F_v	S_{ms}	S_{m1}	S_{ds}	S_{d1}
1.307	0.508	1.00	1.50	1.307	0.762	0.871	0.508

B. Preparation of Soils for Slab On-Grade Improvements

2. *Clearing and Stripping:* Vegetation and improvements should be removed prior to the preparation of the building pad for areas to receive new additions or improvements. This includes any roots from existing trees and shrubbery. Holes resulting from the removal of root systems or other buried obstructions that extend below the planned grades should be cleared and backfilled with properly compacted fill. Shoring will be required near the south property line before deep excavations are made.

3. *Treatment of Existing Fill Soils or Loose Soils:* Should new shallow foundations be desired to support structure, all existing fills should be removed and recompacted down to firm natural soils or the foundations should be deepened to penetrate at least 12 inches into these soils if continuous footings are used (or 6 feet into formational soils if drilled caissons are used), and suspended floors should be designed to span between foundations.

The anticipated depth of removal for the east side additions is approximately 1 to 2 feet. The anticipated depth of removal for the west deck area is a maximum of 2 feet (if constructed on a concrete slab with shallow footings). The recompaction of the existing fill soils should consist of (a) removing these soils down to native medium dense fill or dense formational materials; (b) scarifying, moisture conditioning, and compacting the exposed natural subgrade soils; and (c) cleaning and replacing the removed material as compacted structural fill. Before any soils are processed, our field representative should evaluate the soils at the bottom of the excavation.

The areal extent and depth required to remove the loose surficial fill soils should be confirmed by our representatives during the excavation work based on their examination of the soils being exposed. The lateral extent of the excavation and recompaction should be at least 5 feet beyond the edge of the perimeter foundations of the new residential additions and any areas to receive exterior improvements where feasible (or a distance equal to the depth of soil removal, if farther than 5 feet and feasible).

Any unsuitable materials (such as oversize rubble or rocks, and/or organic matter) should be selectively removed as directed by our representative and disposed of off-site. Any rigid improvements founded on the existing loose or soft surface soils can be expected to undergo movement and possible damage. ***Geotechnical Exploration, Inc.*** takes no responsibility for the performance of any improvements built on loose natural soils or inadequately compacted fills. Subgrade soils in any exterior area receiving concrete improvements should be verified for compaction and moisture within 48 hours prior to concrete placement.

4. *Subgrade Preparation:* After areas to receive new additions/improvements have been cleared, stripped, and the required excavations made, the exposed subgrade soils in areas to receive fill and/or building improvements should be scarified to a depth of 6 inches, moisture conditioned, and compacted to the requirements for structural fill. The near-surface moisture content of clayey soils should be maintained by periodic sprinkling until within 48 hours prior to concrete placement.

5. *Expansive Soil Conditions:* We do not anticipate that significant quantities of highly expansive clay soils will be encountered during grading. Encountered clayey fill soils are of generally high moisture content. Should such soils (of lower moisture content) be encountered and used as fill, however, they should be moisture conditioned or dried to no greater than 5 percent (and not less than 3 percent) above Optimum Moisture content, compacted to 88 to 92 percent, except behind retaining walls. Soils of medium or greater expansion potential should not be used as retaining wall backfill soils except behind shoring walls where a higher soil pressure is recommended.

6. *Material for Fill:* Any required imported fill material should be a low-expansion potential (Expansion Index of 50 or less per ASTM D4829-11). In addition, both imported and existing on-site materials for use as fill should not contain rocks or lumps more than 6 inches in greatest dimension. All materials for use as fill should be approved by our firm prior to filling.

7. *Fill Compaction:* All structural fill to receive the new foundations and slabs should be compacted to a minimum degree of compaction of 90 percent based upon ASTM D1557-12. Fill material should be spread and compacted in uniform horizontal lifts not exceeding 8 inches in uncompacted thickness.

Before compaction begins, the fill should be brought to a moisture content that will permit proper compaction by either: (1) aerating and drying the fill if it is too wet, or (2) moistening the fill with water if it is too dry. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of moisture. For low expansive soils, the moisture content should be within 2 percent of optimum. As previously indicated, medium to highly expansive soils should be moisture conditioned to at least 3 percent above Optimum Moisture content.

As an alternative to fill soil recompaction, deepened foundations and raised wood floors or structural slabs may be considered.

No uncontrolled fill soils should remain on the site after completion of the site work. In the event that temporary ramps or pads are constructed of uncontrolled fill soils, the loose fill soils should be removed and/or recompacted prior to completion of the grading operation.

8. *Trench and Retaining/Basement Wall Backfill:* Utility trenches and retaining walls should preferably be backfilled with compacted fill; gravel is also a suitable backfill material but should be used only if space constraints will not allow the use of compaction equipment. Gravel can also be used as backfill around perforated subdrains. All backfill material should be placed in lift thicknesses appropriate to the type of compaction equipment utilized and compacted to a minimum degree of compaction of 90 percent by mechanical means.

Our experience has shown that even shallow, narrow trenches (such as for irrigation and electrical lines) that are not properly compacted can result in problems, particularly with respect to shallow groundwater accumulation and migration. All trenches and/or narrow areas that are backfilled with gravel should be provided with a properly compacted soil cap layer at least 8 inches thick.

Backfill soils placed behind retaining walls should be installed as early as the retaining walls are capable of supporting lateral loads. Backfill soils behind retaining walls should be low expansive, with an Expansion Index equal to or lower than 50. All areas backfilled with gravel should be capped with a 12-inch-thick layer of properly compacted on-site soils. A Mirafi 140N geofabric should be used to separate soils from crushed rock gravel behind retaining walls and in areas protecting perforated subdrains.

C. Design Parameters for Proposed Foundations

9. *Deepened Footings:* If the existing surface soils are not removed and recompacted, deepened footings for proposed foundation should be founded at least 24 below the lowest adjacent finished grade, have a minimum width of 15 inches, and penetrate at least 1½ feet into dense or very firm formational soils. The deepened footings should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. The final dimensions and reinforcing should be specified by the structural engineer. A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing. If structural floor slabs or wood floors are used, they should be designed to

span the distance between continuous footings. If drilled caissons or piers will be considered, additional recommendations may be provided by our firm.

NOTE: The project Civil/Structural Engineer should review all reinforcing schedules. The reinforcing minimums recommended herein are not to be construed as structural designs, but merely as minimum reinforcement to reduce the potential for cracking and separations.

10. *Slope Top Footings*: All footings located closer than 8 feet inside the top or face of a slope should be deepened to 1½ feet below a line beginning at a point 8 feet horizontally inside the slope and projected outward and downward, parallel to the face of the slope and into firm soils (see Figure No. VIII).

    Bearing surfaces for footings located adjacent to utility trenches should be situated below an imaginary 1.0:1.0 plane projected upward from the bottom edge of the adjacent utility trench. Otherwise, the trenches should be excavated farther from the footing locations.

11. *Shallow Footings*: Shallow footings should bear on undisturbed dense or very firm formational materials or properly compacted fill soils. The footings should be founded at least 18 inches below the lowest adjacent finished grade when founded into properly compacted fill (or 18 inches into formational material). Footings located adjacent to utility trenches should have their bearing surfaces situated below an imaginary 1.5:1.0 plane projected upward from the bottom edge of the adjacent utility trench.

12. *Bearing Values*: At the recommended depths, footings on native, medium dense formational soil or properly compacted fill soil may be designed for allowable bearing pressures of 2,500 pounds per square foot (psf) for combined dead and live loads and may be increased one-third for all loads, including wind or seismic. The footings should have a minimum width of 12 inches. Footings for the western deck (underlain by 2 feet of recompacted fill) should be deepened to penetrate into dense formational soils or properly compacted fills as indicated above, and may be increased one-third when including seismic or wind loading.

13. *Footing Reinforcement*: All continuous footings should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. We recommend that a minimum of two No. 5 top and two No. 5 bottom reinforcing bars be provided in the footings. A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing. Isolated square footings should contain, as a minimum, a grid of three No. 4 steel bars on 12-inch centers, both ways. In order for us to offer an opinion as to whether the footings are founded on soils of sufficient load bearing capacity, it is essential that our representative inspect the footing excavations prior to the placement of reinforcing steel or concrete.

    NOTE: The project Civil/Structural Engineer should review all reinforcing schedules. The reinforcing minimums recommended herein are not to be construed as structural designs, but merely as minimum reinforcement to reduce the potential for cracking and separations.

14. *Lateral Loads:* Lateral load resistance for structure foundations may be developed in friction between the foundation bottoms and the supporting subgrade. An allowable friction coefficient of 0.42 is considered applicable. An additional allowable passive resistance equal to an equivalent fluid weight of 300 pounds per cubic foot acting against the foundations may be used in design provided the footings are poured neat against the adjacent undisturbed formational materials and/or properly compacted fill materials.

    In areas where existing, inadequately compacted fill soils are present in front of foundations (i.e., within 3 times the depth of embedment), the allowable passive resistance should be reduced to 150 pcf and friction coefficient to 0.35. These lateral resistance values assume a level surface in front of the footing for a minimum distance of three times the embedment depth of the footing.

15. *Settlement:* Settlements under building loads are expected to be within tolerable limits for the proposed structure. For footings designed in accordance with the recommendations presented in the preceding paragraphs, we anticipate that total settlements should not exceed 1 inch and that post-construction differential angular rotation should be less than 1/240.

D. Caisson Recommendations

The following recommendations are provided for use by the Structural Engineer in design of the foundations.

16. *Deepened Continuous Footings:* If deepened continuous footings are utilized in areas of relatively shallow fill, they may be deepened to penetrate at least 18 inches into dense formational soils measured on the downhill side of the formational soils slope. The allowable soil end bearing capacity of shallow footings bearing into firm or dense formational soils is 2,500 psf.

17. *Caisson-supported Grade Beam Footings:* Grade beam footings should be founded at least 18 inches below the lowest adjacent finished grade and should have a minimum width of 18 inches. The grade beam footings should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. The final dimensions and reinforcing should be specified by the structural engineer based on the spacing of the caissons as well as load per caissons. A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing.

    NOTE: The project Civil/Structural Engineer should review all reinforcing schedules. The reinforcing minimums recommended herein are not to be construed as structural designs, but merely as minimum reinforcement to reduce the potential for cracking and separations.

18. *Caisson Design*: Where caissons are utilized, they should be designed by the project Civil/Structural Engineer to support all vertical and lateral loads of the proposed structures and/or exterior primary rigid improvements (e.g., proposed retaining walls, swimming pool and spa, carport structures, etc.) where applicable.

19. *End-bearing Caissons*: For vertical loading, all end-bearing caissons should be embedded at least 10 feet into dense (very stiff) formational materials (through the existing fill soils and any top soil/or slopewash if encountered).

    When drilling excavations for caissons utilizing end-bearing strength, it is important to limit the amount of loose material at the bottom of the excavation. Therefore, we recommend that caissons be designed with a minimum diameter of 2 feet in order to facilitate observation of the excavations and allow ease of material removal at the bottom. No slough over 1 inch in thickness should remain at the bottom of the excavation before concrete placement. The drilling contractor should provide an appropriate cleaning tool to satisfy this requirement. Otherwise, shoring installation and hand-tool cleaning (or another acceptable option) will be required. The maximum depth of end-bearing caissons is estimated to be about 40 feet, beneath proposed basement level. The caisson spacing will depend upon the structural designer's choice for grade-beam and slab dimensions as well as design loads.

20. *Vertical Caisson Bearing Capacity*: The recommended allowable end bearing capacity is 20,000 psf for caissons penetrating at least 10 feet into dense (very stiff) formational soils and at least 15 feet below the soil surface when existing fills are present. This end-bearing capacity has already deducted the downdrag force produced by existing fills. The caisson weight to be considered is only one-third the actual weight of the buried caisson. For any exposed portion of caisson, the weight to be considered is 150 pcf. The actual required caisson length and embedment into formational soils should be established by the structural engineer based on the length required to adequately support the total vertical and lateral loads included in the design.

An average allowable increase of 550 psf of shaft frictional capacity can be used for caissons embedded at least 10 feet into formational soils (and at least 15 feet below the ground surface and at least 10 feet into formation).

The recommended allowable end-bearing vertical capacity already includes the effect of negative friction produced by the existing fills. Any caisson weight (150 pcf) above the soil surface should be considered as dead load and should be deducted from the net end-bearing capacity. Caisson depth for the lower-level basement or shallow footings into formation should not be shorter than 10 feet. Due to fill thickness, actual total length may vary at other locations.

21. *Minimum Caisson Spacing*: The minimum center-to-center spacing of caissons in a perpendicular direction to the temporary seismic or wind lateral load should be 3 caisson diameters. For caissons paralleling seismic or wind lateral loads, the shadow effect produces a reducing effect in combined individual lateral load capacity. Therefore, the caisson reduction multiplier for caisson diameters of 3B, 4B, 5B, 6B and 7B (where B is in feet) is 0.6, 0.75, 0.9, 1.0, 1.0, respectively, for leading row caissons; and 0.4, 0.6, 0.75, 0.9, and 1.0 for trailing row caissons.

22. *Lateral Resistance*: For lateral earthquake or wind load resistance, the structural engineer may use any method that considers the equilibrium of forces and moments. For caissons near the slope top, the effective depth for seismic or wind loading resistance should be vertically measured from the horizontal plane providing a setback of 8 feet to daylight. For static loading, we also recommend that caissons closer than 8 feet to the slope top or slope face be designed to support a lateral soil load directed to the slope face. This

soil lateral load will be zero for caissons located at distance beyond 8 feet from slope top, and the maximum soil lateral load will be for caissons located within 8 feet of the slope top. The load should be calculated as active soil pressure (triangular distribution) ranging from zero for caissons at least 8 feet behind the slope top, to the maximum soil pressure for caissons on the slope face or at the top of the slope, with an equivalent fluid weight of 55 pcf acting on twice the caisson diameter and the varying depth, depending on the caisson's distance from the top of the slope. The maximum depth to apply this active pressure is 8 feet.

Soil passive resistance for caissons should be considered starting 8 feet below the ground surface at the top of the slope and 3 feet below the surface for caissons behind 8 feet away from the slope top.

If a balance of forces is calculated based on the applied lateral forces and reaction soil forces, the following allowable passive (equivalent fluid) forces are recommended: 150 pcf for existing fill and 300 pcf for formational soils or properly compacted fill. The passive resistance should be measured from where the depth of caissons is at least 8 feet to the slope face. The passive resistance of the caissons may be considered applicable on a projected surface equal to 2½ times the diameter of the caisson multiplied by the vertical length of embedment being considered. For caissons near slope faces, passive resistance against seismic or wind loading may start to be measured from a horizontal plane providing a setback distance of 8 feet to the slope face.

23. *Caisson Drilling Observations*: Caisson drilling or excavation operations should be performed under the continued observations of a representative of our firm to confirm the penetration into formational soils.

24. *Caisson Design Standards:* The design and construction of the caissons should be in accordance with the recommendations presented above, the current CBC requirements accepted by the City of San Diego, and also in accordance with ACI 336, 4R-93 Design and Construction of Drilled Piers, of the American Concrete Institute. The contractor shall follow all the safety procedures required by Cal OSHA.

E. Concrete Slab On-grade Criteria

Slabs on-grade may only be used on new, properly compacted fill or when bearing on dense natural soils. If concrete slabs are planned on existing fills, they should be designed as structural slabs spanning between foundations bearing in formational soils.

25. *Minimum Floor Slab Reinforcement:* Based on our experience, we have found that, for various reasons, floor slabs occasionally crack. Therefore, we recommend that all slabs-on-grade contain at least a minimum amount of reinforcing steel to reduce the separation of cracks, should they occur. Slab subgrade soil should be verified by a ***Geotechnical Exploration, Inc***. representative to have the proper moisture content within 48 hours prior to placement of the vapor barrier and pouring of concrete.

New interior floor slabs should be a minimum of 5 inches actual thickness and be reinforced with No. 4 bars on 18-inch centers, both ways, placed at midheight in the slab. *The slabs should be underlain by a 2-inch-thick layer of clean sand (S.E. = 30 or greater) overlying a moisture retardant membrane over 2 inches of sand.* Slab subgrade soil should be verified by a ***Geotechnical Exploration, Inc.*** representative to have the proper moisture content within 48 hours prior to placement of the vapor barrier and pouring of concrete.

26. *Slab Moisture Protection and Vapor Barrier Membrane:* Although it is not the responsibility of geotechnical engineering firms to provide moisture protection recommendations, as a service to our clients we provide the following discussion and suggested minimum protection criteria. Actual recommendations should be provided by the project architect and waterproofing consultants or product manufacturer.

Soil moisture vapor can result in damage to moisture-sensitive floors, some floor sealers, or sensitive equipment in direct contact with the floor, in addition to mold and staining on slabs, walls, and carpets. The common practice in Southern California is to place vapor retarders made of PVC, or of polyethylene. PVC retarders are made in thickness ranging from 10- to 60-mil. Polyethylene retarders, called visqueen, range from 5- to 10-mil in thickness. These products are no longer considered adequate for moisture protection and can actually deteriorate over time.

Specialty vapor retarding and barrier products possess higher tensile strength and are more specifically designed for and intended to retard moisture transmission into and through concrete slabs. The use of such products is highly recommended for reduction of floor slab moisture emission.

The following American Society for Testing and Materials (ASTM) and American Concrete Institute (ACI) sections address the issue of moisture transmission into and through concrete slabs: ASTM E1745-97 (2009) Standard Specification for Plastic Water Vapor Retarders Used in Contact Concrete Slabs; ASTM E154-88 (2005) Standard Test Methods for Water Vapor Retarders Used in Contact with Earth; ASTM E96-95 Standard Test Methods for Water Vapor Transmission of Materials; ASTM E1643-98 (2009) Standard Practice for Installation of Water Vapor Retarders Used in Contact Under Concrete Slabs; and ACI 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials.

26.1 Based on the above, we recommend that the vapor barrier consist of a minimum 15-mil extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditioning (ASTM E1745 Section 7.1 and sub-paragraphs 7.1.1-7.1.5) should be less than 0.01 U.S. perms (grains/square foot/hour/inch of mercury [Hg]) and comply with the ASTM E1745 Class A requirements. Installation of vapor barriers should be in accordance with ASTM E1643. The basis of design is Stego wrap vapor barrier 15-mil. The vapor barrier should be placed in accordance with the manufacturer's specifications.

26.2 Common to all acceptable products, vapor retarder/barrier joints must be lapped and sealed with mastic or the manufacturer's recommended tape or sealing products. In actual practice, stakes are often driven through the retarder material, equipment is dragged or rolled across the retarder, overlapping or jointing is not properly implemented, etc. All these construction deficiencies reduce the retarder's effectiveness. In no case should retarder/barrier products be punctured or gaps be allowed to form prior to or during concrete placement.

26.3 Following placement of concrete floor slabs, sufficient drying time must be allowed prior to placement of any floor coverings. Premature placement of floor coverings may result in degradation of adhesive materials and loosening of the finish floor materials.

27. *Concrete Isolation Joints:* We recommend the project Civil/Structural Engineer incorporate isolation joints and sawcuts to at least one-fourth the thickness of the slab in any floor designs. The joints and cuts, if properly placed, should reduce the potential for and help control floor slab cracking. We recommend that concrete shrinkage joints be spaced no farther than approximately 20 feet apart, and also at re-entrant corners. However, due to a number of reasons (such as base preparation, construction techniques, curing procedures, and normal shrinkage of concrete), some cracking of slabs can be expected.

The new western concrete deck can be isolated from the foundation of the home by ½-inch felt provided with flexible caulking material at the top.

28. *Exterior Slab Reinforcement:* Exterior concrete slabs should be at least 4 inches thick. As a minimum for protection of on-site improvements, we recommend that all nonstructural concrete slabs (such as patios, sidewalks, etc.), be founded on properly compacted and tested fill or dense native

formation and be underlain by 2 inches and no more than 3 inches of clean leveling sand, with No. 3 bars at 18-inch centers, both ways, at the center of the slab. Exterior slabs should contain adequate isolation and control joints.

The performance of on-site improvements can be greatly affected by soil base preparation and the quality of construction. It is therefore important that all improvements are properly designed and constructed for the existing soil conditions. The improvements should not be built on loose soils or fills placed without our observation and testing. The subgrade of exterior improvements should be verified as properly prepared within 48 hours prior to concrete placement. A minimum thickness of 2 feet of properly recompacted soils should underlie the exterior slabs on-grade or be built on dense formational soils.

29. *Exterior Slab Control Joints*: For exterior slabs with the minimum shrinkage reinforcement, control joints should be placed at spaces no farther than 12 feet apart or the width of the slab, whichever is less, and also at re-entrant corners. Control joints in exterior slabs should be sealed with elastomeric joint sealant. The sealant should be inspected every 6 months and be properly maintained. Concrete slab joints should be dowelled or continuous steel reinforcement should be provided to help reduce any potential differential movement.

F. ***Retaining/Basement Wall Design Criteria***

30. *Static Design Parameters:* Retaining/Basement walls must be designed to resist lateral earth pressures and any additional lateral pressures caused by surcharge loads on the adjoining retained surface. We recommend that

restrained retaining walls with level backfill be designed for an equivalent fluid pressure of 65 pcf for existing soils or 56 pcf for low expansive soils (import). Wherever restrained walls will be subjected to surcharge loads, they should also be designed for an additional uniform lateral pressure equal to 0.58 times the anticipated surcharge pressure for on-site clayey soils and 0.47 times the anticipated surcharge pressure for expansive import soils. For unrestrained walls with on-site expansive level backfill, the coefficient is 0.42 (and 0.31 for imported, low-expansive soils).

Exterior unrestrained retaining walls supporting a 2.0:1.0 (h:v) backfill may be designed for an equivalent fluid weight of 52 pcf (using low expansive soils) and 65 pcf for on-site expansive soils with a 2.0:1.0 (h:v) sloping backfill. Restrained retaining walls supporting a 2.0:1.0 (h:v) backfill of low expansive soils should be designed with a soil pressure of 75 pcf and 90 pcf for on-site expansive soils supporting a 2.0:1.0 (h:v) sloping backfill.

Backfill placed behind the walls should be compacted to a minimum degree of compaction of 90 percent using light compaction equipment. If heavy equipment is used, the walls should be appropriately temporarily braced.

31. *Retaining Wall Seismic Earth Pressures*: If seismic loading is considered for retaining walls more than 6 feet in height, they should be designed for seismic earth pressures in addition to the normal static pressures. For the retaining wall (restrained) with level backfill, we recommend that the seismic pressure increment be taken as an additional fluid pressure distribution (zero pressure at the ground surface and maximum pressure at the base) utilizing an equivalent fluid weight of 16 pcf. A Kh value of 0.18 may be used is a computer program such as "*Retaining Wall Pro*" or a similar program is used

for wall design. The soil pressure described above may be used for the design of shoring structures.

32. *Cal-OSHA*: Where not superseded by specific recommendations presented in this report, trenches, excavations, and temporary slopes at the subject site should be constructed in accordance with Title 8, Construction Safety Orders, issued by Cal-OSHA.

G. Keystone (Segmental Retaining Wall) Recommendations

Keystone retaining walls are proposed to be placed on the lower portion of the rear yard slope to create access and usable yard area on the lower level of the rear yard. Refer to Figure No. II for a representation of the approximate wall location.

33. *Geogrid Reinforcement*: Fill soils placed behind segmental block walls must be reinforced with geogrid layers typically placed horizontally every 24 inches in depth beginning at the top of the bottom row block. All fill soil should be properly compacted. The geogrid should extend from the front of the wall connection to the back of the excavation complying with the specified length. The segmental wall designer should specify the type of block, tilt, geogrid, geogrid spacing, length, etc.

34. *Geotechnical Parameters for Segmental Retaining Walls*: Based on information obtained from our subsurface investigation, we recommend the following geotechnical parameters for design of the segmental retaining walls.

Parameter	**Reinforced Zone (Import)**	**Retained Zone (On-site)**	**Foundation Zone (On-site)**
Angle of Internal Friction	32 degrees*	28 degrees	28 degrees
Cohesion	100 psf	200 psf	200 psf
Wet Unit Weight	120 pcf	120 pcf	120 pcf

**Based on laboratory testing and our experience, it is our opinion that this is an acceptable parameter value. However, laboratory testing of the backfill material will be required prior to construction to confirm this value.*

The soil parameters provided in the table above are based on grain size analysis tests performed and represent typical granular on-site materials. It is the responsibility of the wall designer to use his judgment in the selection of design parameters. If import materials will be used for wall backfill, sufficient shear tests should be conducted on samples of the proposed backfill materials to verify they conform to actual design values. Results should be provided to the designer to re-evaluate stability of the walls if the shear test results differ from assumed or specified soil values. Dependent upon test results, the designer may require modifications to the original wall design (i.e., longer geogrid embedment lengths or closer vertical spacing of the geogrid layers).

The above parameters also assume that the walls will be founded on properly compacted fill materials. Compacted fill is also expected to comprise the retained and reinforced zones. The foundation zone is the area where the footing is embedded; the reinforced zone is the area of the backfill that possesses the reinforcing fabric; and the retained zone is the area behind the reinforced zone.

35. *Wall Backfill*: Backfill materials within the reinforced zone should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density at or slightly above optimum moisture content in accordance with ASTM D1557-09. This is applicable to the entire embedment width of the geogrid reinforcement. Typically, wall designers specify no heavy compaction equipment within 3 feet of the face of the wall. However, smaller equipment (i.e., walk-behind, self-driven compactors or hand whackers) can be used to compact the materials without causing deformation of the wall. If the designer specifies no compactive effort for this zone, then the materials are essentially not properly compacted and the geogrid within the uncompacted zone should not be relied upon for reinforcement. Therefore, overall embedment lengths must be increased to account for the difference. The recommended minimum setback distance from the bottom block to the slope face is 6 feet.

36. *Movement of Geogrid Reinforced Soils*: Geosynthetic reinforcement must elongate to develop full tensile resistance. This elongation generally results in some degree of movement at the top of the wall. The amount of movement is dependent upon the height of the wall (i.e., higher walls rotate more) and the type of geogrid reinforcing used. In addition, over time geogrid has been known to exhibit creep and can undergo additional movement. Given this condition, the owner should be aware that structures placed within the reinforced and retained zones of the wall may undergo movement. The contractor must stretch the geogrid and place stakes or nails to hold it taut before backfill soils are placed and compacted on each layer of geogrid. The structural designer of the wall must consider the potential deformation of the geogrid to recommend a conservative allowable strength of the geogrid.

H. Wall Drainage Recommendations

37. *Wall Drainage:* The preceding design pressures assume that the walls are backfilled with the on-site soils or imported low-expansive soils, and that there is sufficient drainage behind the walls to prevent the build-up of hydrostatic pressures from surface water infiltration. We recommend that drainage be provided by a composite drainage material such as Miradrain 6000/6200 or equivalent. The drain material should terminate 3 inches below the finish surface where the surface is covered by pavements or slabs or 6 inches below the finish surface in landscape areas (see Figure No. IX for Retaining Wall Drainage schematic). Waterproofing should extend from the bottom to the top of the wall.

    Geotechnical Exploration, Inc. will assume no liability for damage to structures or improvements that is attributable to poor drainage. The architectural plans should clearly indicate that subdrains for any lower-level walls be placed at an elevation at least 1 foot below the bottom of the lower-level slabs. At least 0.5-percent gradient should be provided to the subdrain. The subdrain should be placed in an envelope of crushed rock gravel up to 1 inch in maximum diameter, and be wrapped with Mirafi 140N filter or equivalent. The collected water should be taken to an approved drainage facility.

38. *Drainage Quality Control*: It must be understood that it is not within the scope of our services to provide quality control oversight for surface or subsurface drainage construction or retaining wall sealing and base of wall drain construction. It is the responsibility of the contractor to verify proper wall sealing, geofabric installation, protection board (if needed), drain depth below interior floor or yard surface, pipe percent slope to the outlet, etc.

I. ***Swimming Pool***

39. *Swimming Pool Recommendations:* It is our understanding that a swimming pool is planned for the northwestern portion of the lot. The swimming pool foundation should be founded entirely in cut native soils and the bottom should be designed as a structural slab. If this is not feasible, then the entire pool shell area should be founded on caissons (refer to Caisson Recommendation section for details). An alternative would be to compact the low-expansive fill soils to at least 95 percent relative compaction under the pool shell. The soils surrounding the swimming pool should be low-expansive. In addition, any above-grade portion of the pool (where applicable) should be designed as a free-standing wall. The swimming pool shell should be designed for a soil pressure of at least 45 pcf (for low expansive soils) if the pool wall is considered a cantilever wall free to rotate; or 56 pcf if considered as a restrained wall with low-expansive backfill. In addition, the outer side of the pool (or spa) should be provided with a foundation setback of at least 8 feet to daylight.

The pool deck subgrade should be properly moisture conditioned and compacted, and should be verified by our firm within 48 hours prior to steel and concrete placement. The pool deck should have dowels or continuous steel reinforcement at all joint locations to help reduce the potential for vertical differential damage. In addition, the control and isolation joints shall be sealed with elastomeric joint sealant. The sealant should be inspected and maintained periodically by the owner. The swimming pool deck and surrounding area should be provided with adequate surface drainage including positive surface drainage and/or functional area drains.

J. Slopes

It is our understanding that no large permanent slopes are proposed. Temporary slopes may be required during site preparation and construction.

40. *Slope Observations:* A representative of ***Geotechnical Exploration, Inc.*** must observe any steep temporary slopes *during construction*. In the event that soils and formational material comprising a slope are not as anticipated, any required slope design changes would be presented at that time.

41. *Permanent Slopes*: Any new cut or fill slopes up to 10 feet in height should be constructed at an inclination of 2.0:1.0 (horizontal to vertical). Permanent slopes at a 2.0:1.0 slope should possess a factor of safety of 1.5 against deep and shallow failure. Refer to Appendix C, Slope Stability Analyses.

42. *Temporary Slopes*: Based on our subsurface investigation work, laboratory test results, and engineering analysis, temporary slopes should be stable for a maximum slope height of up to 12 feet and may be cut at a slope ratio of 0.5:1.0 in properly compacted fill soils and in dense natural soils. Some localized sloughing or raveling of the soils exposed on the slopes, however, may occur. No surcharge should exist or be placed behind temporary cut slopes.

    Since the stability of temporary construction slopes will depend largely on the contractor's activities and safety precautions (storage and equipment loadings near the tops of cut slopes, surface drainage provisions, etc.), it should be the contractor's responsibility to establish and maintain all temporary construction slopes at a safe inclination appropriate to the method of operation. No soil stockpiles or surcharge may be placed within a horizontal distance of 10 feet from the excavation.

If these recommendations are not feasible due to space constraints, temporary shoring may be required for safety and to protect adjacent property improvements. Similarly, footings near temporary cuts should be underpinned or protected with shoring. On-site expansive soil values given for retaining walls and caissons are applicable for the shoring design.

K. ***Site Drainage Considerations***

43. *Erosion Control*: Appropriate erosion control measures should be taken at all times during and after construction to prevent surface runoff waters from entering footing excavations or ponding on finished building pad areas.

44. *Surface Drainage:* Adequate measures should be taken to properly finish-grade the lot after the additions and other improvements are in place. Drainage waters from this site and adjacent properties should be directed away from the footings, floor slabs, and slopes, onto the natural drainage direction for this area or into properly designed and approved drainage facilities provided by the project civil engineer. Roof gutters and downspouts should be installed on the residence, with the runoff directed away from the foundations via closed drainage lines. Proper subsurface and surface drainage will help minimize the potential for waters to seek the level of the bearing soils under the footings and floor slabs.

Failure to observe this recommendation could result in undermining and possible differential settlement of the structure or other improvements on the site or cause other moisture-related problems. Currently, the CBC requires a minimum 1-percent surface gradient for proper drainage of building pads unless waived by the building official. Concrete pavement may have a minimum gradient of 0.5-percent.

45. *Planter Drainage:* Planter areas, flower beds and planter boxes should be sloped to drain away from the footings and floor slabs at a gradient of at least 5 percent within 5 feet from the perimeter walls. Any planter areas adjacent to the residence or surrounded by concrete improvements should be provided with sufficient area drains to help with rapid runoff disposal. No water should be allowed to pond adjacent to the residence or other improvements or anywhere on the site.

L. General Recommendations

46. *Project Start Up Notification:* In order to reduce work delays during site development, this firm should be contacted 48 hours prior to any need for observation of footing excavations or field density testing of compacted fill soils. If possible, placement of formwork and steel reinforcement in footing excavations should not occur prior to observing the excavations; in the event that our observations reveal the need for deepening or redesigning foundation structures at any locations, any formwork or steel reinforcement in the affected footing excavation areas would have to be removed prior to correction of the observed problem (i.e., deepening the footing excavation, recompacting soil in the bottom of the excavation, etc.).

47. *Construction Best Management Practices (BMPs):* Construction BMPs must be implemented in accordance with the requirements of the controlling jurisdiction. Sufficient BMPs must be installed to prevent silt, mud or other construction debris from being tracked into the adjacent street(s) or storm water conveyance systems due to construction vehicles or any other construction activity. The contractor is responsible for cleaning any such debris that may be in the street at the end of each work day or after a storm event that causes breach in the installed construction BMPs.

    All stockpiles of uncompacted soil and/or building materials that are intended to be left unprotected for a period greater than 7 days are to be provided with erosion and sediment controls. Such soil must be protected each day when the probability of rain is 40% or greater. A concrete washout should be provided on all projects that propose the construction of any concrete improvements that are to be poured in place. All erosion/sediment control devices should be maintained in working order at all times. All slopes that are created or disturbed by construction activity must be protected against erosion and sediment transport at all times. The storage of all construction materials and equipment must be protected against any potential release of pollutants into the environment.

XII. GRADING NOTES

Geotechnical Exploration, Inc. recommends that we be retained to verify the actual soil conditions revealed during site grading work and footing excavation to be as anticipated in this "*Update Report of Limited Geotechnical Investigation*" for the project. In addition, the compaction of any fill soils placed during site grading work must be observed and tested by the soil engineer.

It is the responsibility of the grading contractor to comply with the requirements on the grading plans as well as the local grading ordinance. All retaining wall and trench backfill should be properly compacted. ***Geotechnical Exploration, Inc.*** will assume no liability for damage occurring due to improperly or uncompacted backfill placed without our observations and testing.

XIII. LIMITATIONS

Our conclusions and recommendations have been based on available data obtained from our field investigation and laboratory analysis, as well as our experience with similar soils and formational materials located in this area of San Diego. Of necessity, we must assume a certain degree of continuity between exploratory excavations and/or natural exposures. It is, therefore, necessary that all observations, conclusions, and recommendations be verified at the time grading operations begin or when footing excavations are placed. In the event discrepancies are noted, additional recommendations may be issued, if required.

The work performed and recommendations presented herein are the result of an investigation and analysis that meet the contemporary standard of care in our profession within the County of San Diego. No warranty is provided.

This report should be considered valid for a period of two (2) years, and is subject to review by our firm following that time. If significant modifications are made to the building plans, especially with respect to the height and location of any proposed structures, this report must be presented to us for immediate review and possible revision.

As stated previously, it is not within the scope of our services to provide quality control oversight for surface or subsurface drainage construction or retaining wall sealing and base of wall drain construction. It is the responsibility of the contractor to verify proper wall sealing, geofabric installation, protection board installation (if needed), drain depth below interior floor or yard surfaces; pipe percent slope to the outlet, etc.

It is the responsibility of the owner and/or developer to ensure that the recommendations summarized in this report are carried out in the field operations and that our recommendations for design of this project are incorporated in the structural plans. We should be retained to review the project plans once they are available, to verify that our recommendations are adequately incorporated in the plans. Additional or modified recommendations may be issued, if warranted, after plan review.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for the safety of personnel other than our own on the site; the safety of others is the responsibility of the contractor. The contractor should notify the owner if any of the recommended actions presented are considered to be unsafe.

The firm of ***Geotechnical Exploration, Inc.*** shall not be held responsible for changes to the physical condition of the property, such as addition of fill soils or changing drainage patterns, which occur subsequent to issuance of this report and the changes are made without our observations, testing, and approval.

Once again, should any questions arise concerning this report, please feel free to contact the undersigned. Reference to our **Job No. 13-10407** will expedite a reply to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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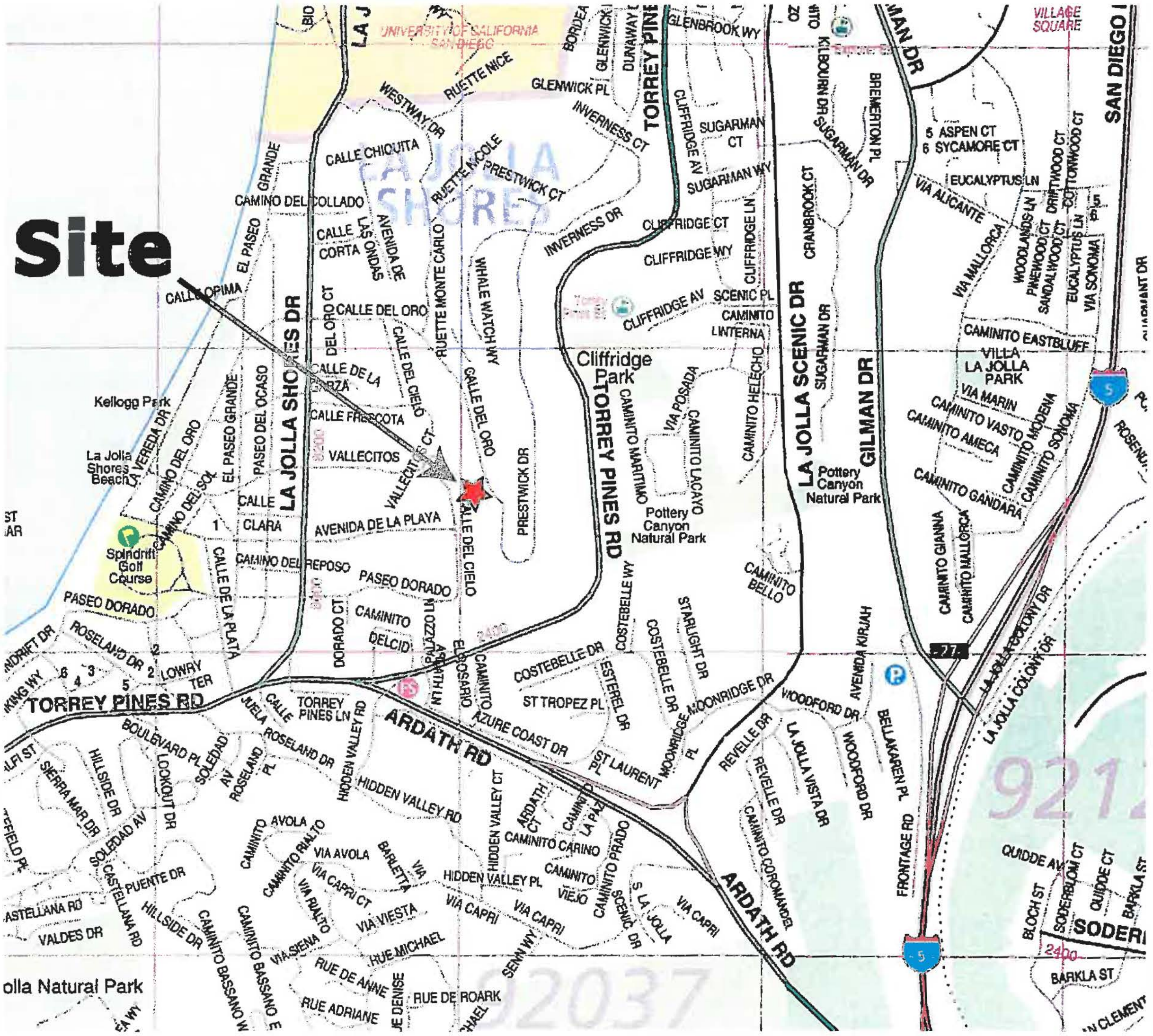
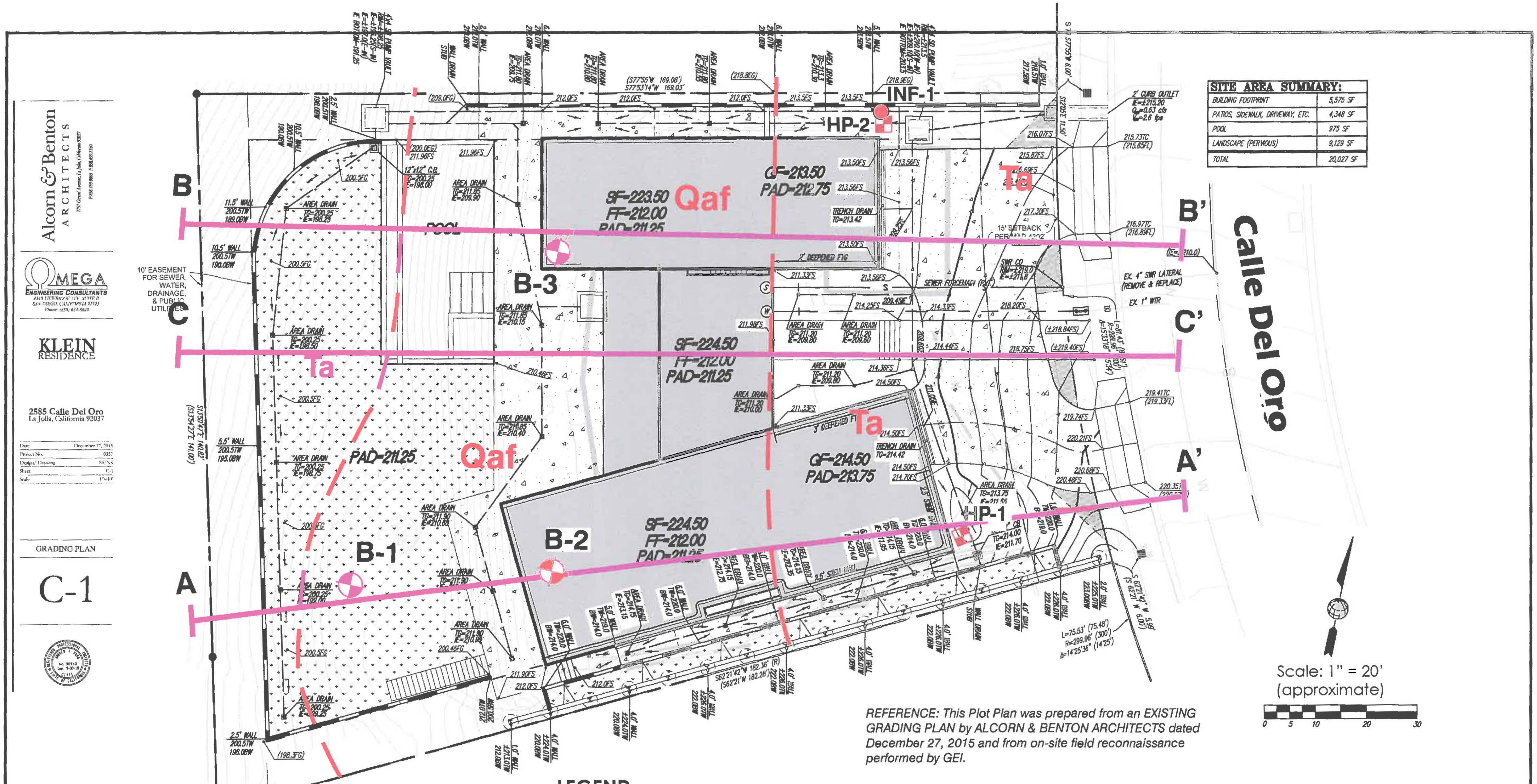
VICINITY MAP



Thomas Bros Guide San Diego County pg 1227-H6

Proposed Klein Residence
2585 Calle Del Oro
La Jolla, CA.

Figure No. I
Job No. 13-10407



SITE AREA SUMMARY:	
BUILDING FOOTPRINT	5,575 SF
PATIOS, SIDEWALK, DRIVEWAY, ETC.	4,348 SF
POOL	975 SF
LANDSCAPE (PERVIOUS)	9,129 SF
TOTAL	20,027 SF

REFERENCE: This Plot Plan was prepared from an EXISTING GRADING PLAN by ALCORN & BENTON ARCHITECTS dated December 27, 2015 and from on-site field reconnaissance performed by GEI.

GEOLOGIC LEGEND

Qaf	Artificial Fill
Ta	Ardath Shale Formation
– – –	Approximate Geologic Contact

LEGEND

B-3	Approximate Location of Exploratory Boring
HP-3	Approximate Location of Exploratory Handpit
● **INF-1**	Approximate Location of Infiltration Test
A — A'	Approximate Line of Cross Section

NOTE: This Plot Plan is not to be used for legal purposes. Locationss and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

PLOT PLAN

Proposed Klein Residence
2585 Calle Del Oro
La Jolla, CA.
Figure No. II
Job No. 13-10407

Geotechnical Exploration, Inc.

July 2016

13-10407-P3.ai

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Limited Access Auger Drill Rig	**6-inch diameter Boring**	**11-1-13**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 207' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
1			**SILTY SAND**, with some clay and brick debris. Loose. Damp. Gray-brown. **FILL (Qaf)**	SM								
2			**SILTY CLAY**, with some caliche and siltstone fragments. Medium dense/ stiff. Dry to damp. Light gray. **FILL (Qaf)**	CL								
3		■			13.3	106.5			91		60	3"
4		▨									23	2"
5			**SANDY SILT**, with some clay and gypsum-filled fractures. Dense/ hard. Damp. Dark gray. **ARDATH SHALE FORMATION (Ta)**	ML								
6												
7												
8		▨									48	2"
9			Bottom @ 8.5'									

- ▼ PERCHED WATER TABLE
- ☒ LOOSE BAG SAMPLE
- 1 IN-PLACE SAMPLE
- ■ MODIFIED CALIFORNIA SAMPLE
- S NUCLEAR FIELD DENSITY TEST
- ▨ STANDARD PENETRATION TEST

JOB NAME
Klein Residence

SITE LOCATION
2585 Calle del Oro, La Jolla, CA

JOB NUMBER	REVIEWED BY	LOG No.
13-10407	**LDR/JAC**	**B-1**
FIGURE NUMBER	Geotechnical Exploration, Inc.	
IIIa		

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Limited Access Auger Drill Rig	**6-inch diameter Boring**	**11-1-13**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 219' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			Lawn grass and topsoil.										
			SILTY CLAY, with sand and siltstone fragments. Loose/ soft. Moist. Light brown and orange.	CL									
			FILL (Qaf)										
2		■			24.3	101.2			87			19	3"
4		▨										6	2"
6		☒	-- 88% passing #200 sieve.				13.5	116.5			68		
		■			21.6	106.7			91			51	3"
8			**SANDY SILT**, with some clay. Dense/ hard. Damp. Dark gray.	ML									
			ARDATH SHALE FORMATION (Ta)										
		▨										56	2"
10													
			Bottom @ 10'										

- ▼ PERCHED WATER TABLE
- ☒ LOOSE BAG SAMPLE
- 1 IN-PLACE SAMPLE
- ■ MODIFIED CALIFORNIA SAMPLE
- S NUCLEAR FIELD DENSITY TEST
- ▨ STANDARD PENETRATION TEST

JOB NAME		
Klein Residence		
SITE LOCATION		
2585 Calle del Oro, La Jolla, CA		
JOB NUMBER **13-10407**	REVIEWED BY **LDR/JAC** **Geotechnical Exploration, Inc.**	LOG No. **B-2**
FIGURE NUMBER **IIIb**		

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Limited Access Auger Drill Rig	**6-inch diameter Boring**	**11-1-13**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 219' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			Lawn grass and topsoil. **SILTY CLAY**, with sand and some siltstone fragments. Loose/ soft. Moist. Light brown and orange. **FILL (Qaf)**	CL								
		■			24.7	93.3			80		8	3"
4		▨									7	2"
8		■			21.1	108.1			93		27	3"
		▨									20	2"
10		▨	**SANDY SILT**, with some clay. Dense/ hard. Damp. Dark gray. **ARDATH SHALE FORMATION (Ta)**	ML								
		▨									40	2"
12			Bottom @ 11.5'									

- ▼ PERCHED WATER TABLE
- ☒ LOOSE BAG SAMPLE
- 1 IN-PLACE SAMPLE
- ■ MODIFIED CALIFORNIA SAMPLE
- S NUCLEAR FIELD DENSITY TEST
- ▨ STANDARD PENETRATION TEST

JOB NAME
Klein Residence

SITE LOCATION
2585 Calle del Oro, La Jolla, CA

JOB NUMBER
13-10407

FIGURE NUMBER
IIIc

REVIEWED BY
LDR/JAC

Geotechnical Exploration, Inc.

LOG No.
B-3

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Hand Tools	**3' X 3' X 3' Handpit**	**7-18-16**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 220' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
1			**CLAYEY SAND**, with abundant roots and rock fragments. Loose to medium dense. Damp. Gray-brown. **FILL (Qaf)**	SC								
2	▨		**SANDY CLAY**, with some gypsum crystals. Stiff to hard. Damp. Light gray and yellow-brown. **ARDATH SHALE FORMATION (Ta)**	CL								
3 4			Bottom @ 3'									

▼ PERCHED WATER TABLE

☒ BULK BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S NUCLEAR FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME
Klein Residence

SITE LOCATION
2585 Calle del Oro, La Jolla, CA

JOB NUMBER
13-10407

REVIEWED BY **LDR/JAC**

Geotechnical Exploration, Inc.

LOG No.
HP-1

FIGURE NUMBER
IIId

EXPLORATION LOG 10407 KLEIN 2016.GPJ GEO_EXPL.GDT 7/25/16

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Hand Tools	**3' X 3' X 3' Handpit**	**7-18-16**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 220' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			CLAYEY SAND, with some roots and rock fragments. Loose to medium dense. Damp. Gray-brown. **FILL (Qaf)**	SC									
1			**SANDY CLAY**, with some gypsum crystals; moderately fractured. Very stiff to hard. Damp. Yellow-brown. **ARDATH SHALE FORMATION (Ta)**	CL									
2		☒	-- 85% passing #200 sieve.				15.5	112.6			48		
			Bedding: N60°W, 3°NE.										
3													
			Bottom @ 3'										
4													

▼ PERCHED WATER TABLE

☒ BULK BAG SAMPLE

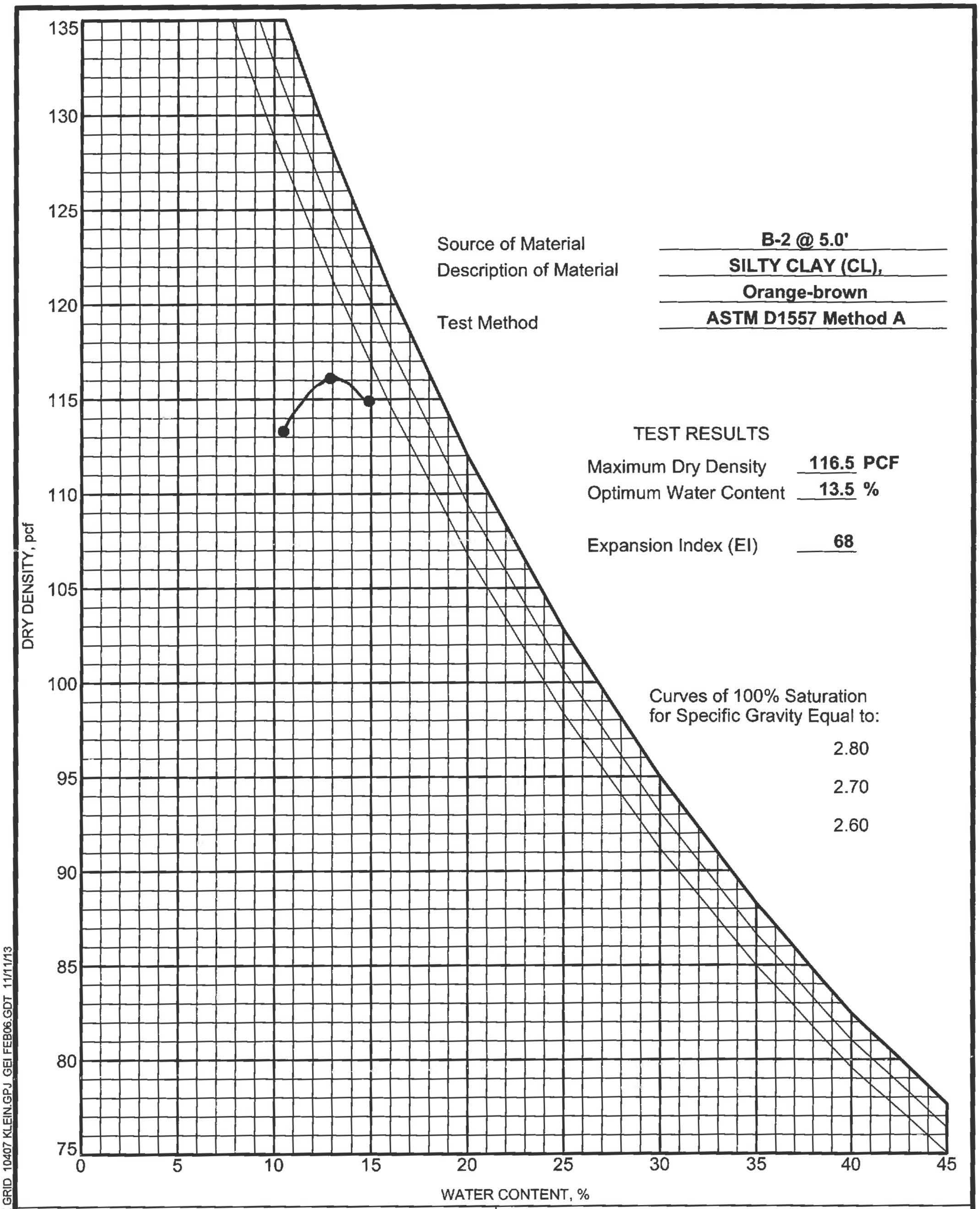
1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S NUCLEAR FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME **Klein Residence**		
SITE LOCATION **2585 Calle del Oro, La Jolla, CA**		
JOB NUMBER **13-10407**	REVIEWED BY **LDR/JAC**	LOG No.
FIGURE NUMBER **IIIe**	**Geotechnical Exploration, Inc.**	**HP-2**



Source of Material **B-2 @ 5.0'**

Description of Material **SILTY CLAY (CL),**
Orange-brown

Test Method **ASTM D1557 Method A**

TEST RESULTS

Maximum Dry Density **116.5 PCF**

Optimum Water Content **13.5 %**

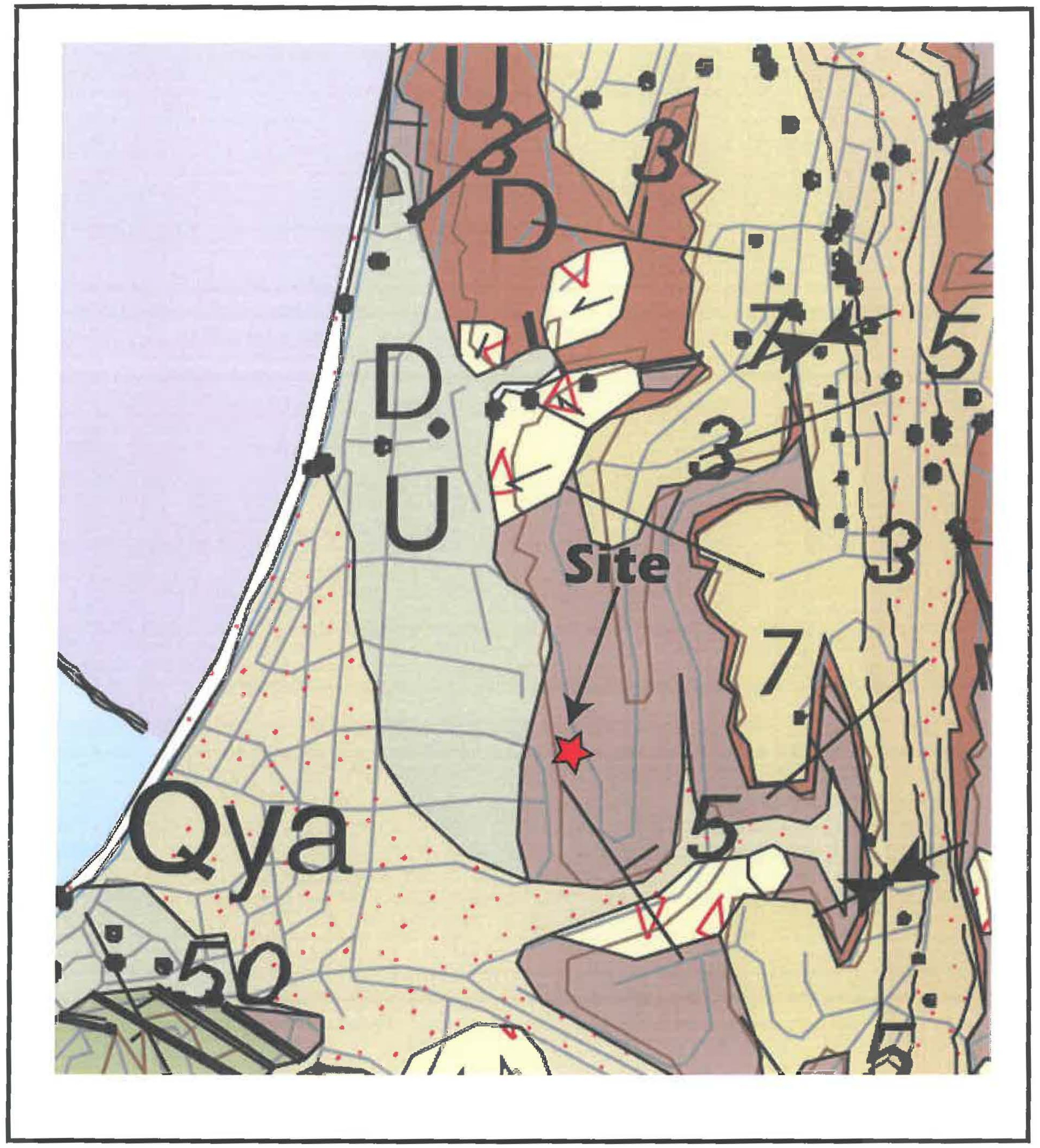
Expansion Index (EI) **68**

Geotechnical Exploration, Inc.

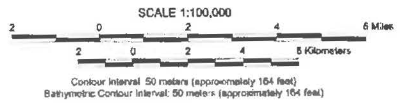
MOISTURE-DENSITY RELATIONSHIP

Figure Number: IV
Job Name: Klein Residence
Site Location: 2585 Calle del Oro, La Jolla, CA
Job Number: 13-10407

COMPACTION + EI DARK GRID 10407 KLEIN.GPJ GEI FEB06.GDT 11/11/13



Klein Residence Additions
2585 Calle Del Oro
La Jolla, CA.



Contour Interval 50 meters (approximately 164 feet)
Bathymetric Contour Interval: 50 meters (approximately 164 feet)

EXCERPT FROM GEOLOGIC MAP OF THE SAN DIEGO 30' x 60' QUADRANGLE, CALIFORNIA

By

Michael P. Kennedy[1] and Siang S. Tan[1]

2008

Digital preparation by

Kelly R. Bovard[2], Anne G. Garcia[2], Diane Burns[2], and Carlos I. Gutierrez[1]

1 Department of Conservation, California Geological Survey
2 U.S. Geological Survey Department of Earth Sciences, University of California, Riverside

ONSHORE MAP SYMBOLS

	Contact - Contact between geologic units; dotted where concealed.
70 U D	Fault - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.
	Anticline - Solid where accurately located; dashed where approximately located; dotted where concealed. Arrow indicates direction of axial plunge.
	Syncline - Solid where accurately located; dotted where concealed. Arrow indicates direction of axial plunge.
Qls?	Landslide - Arrows indicate principal direction of movement. Queried where existence is questionable.

Strike and dip of beds

70	Inclined

Strike and dip of igneous joints

60	Inclined
	Vertical

Strike and dip of metamorphic foliation

55	Inclined

DESCRIPTION OF MAP UNITS

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)

Base Map

Onshore base (hypsography, hydrography and transportation) from U.S.G.S. digital line graph (DLG) data, San Diego 30' x 60' metric quadrangle. Shaded topographic base from U.S.G.S. digital elevation models (DEM's). Offshore bathymetric contours and shaded bathymetry from N.O.A.A. single and multibeam data. Projection is UTM zone 11, North American Datum 1927.

This map was funded in part by the U.S. Geological Survey National Cooperative Geologic Mapping Program STATEMAP Award no. 98HQAG2049

Prepared in cooperation with the U.S. Geological Survey, Southern California Areal Mapping Project.

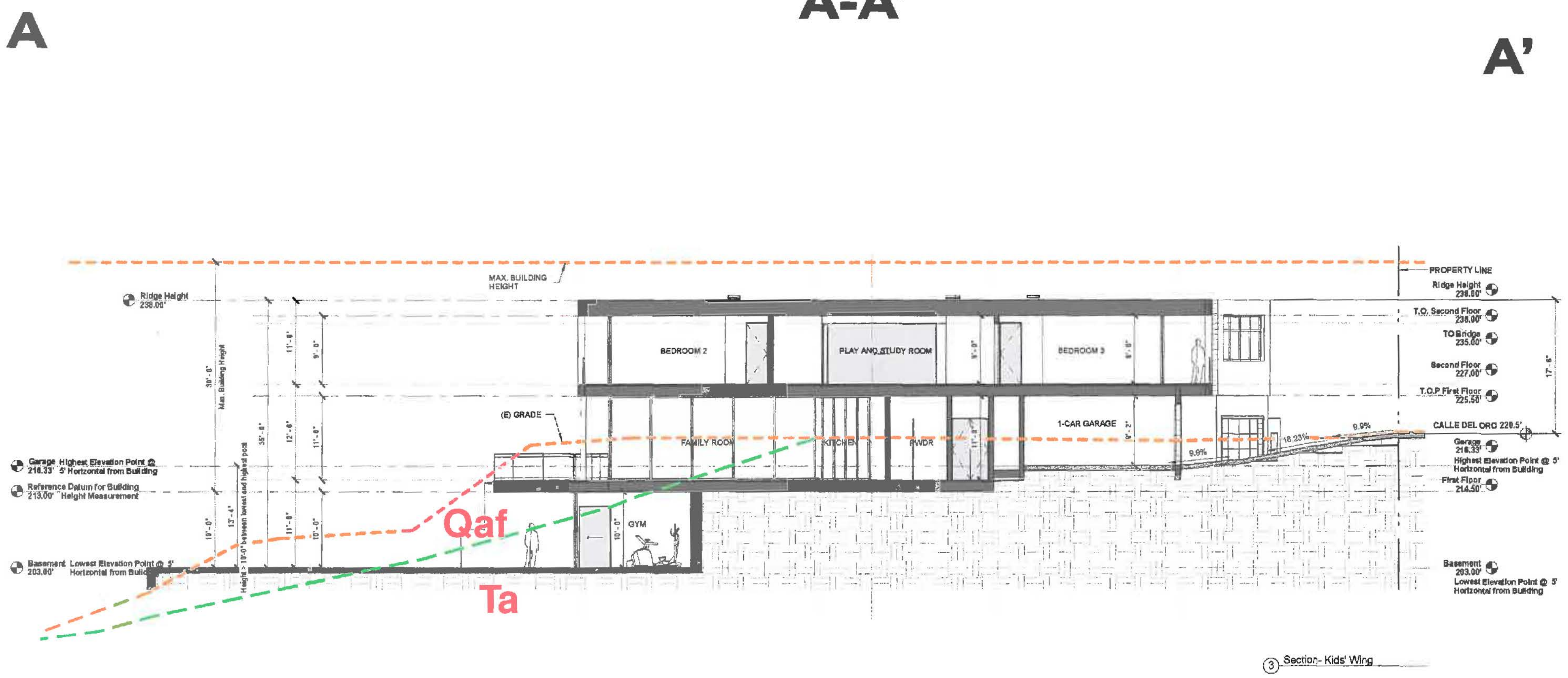
Figure No. V
Job No. 13-10407

Geotechnical Exploration, Inc.

July 2016

klein-2008-geo.ai

CROSS SECTION
A-A'



Scale: 1" = 20'
(approximate)

REFERENCE: This Cross Section was prepared from an existing SECTION PLANS by Studio William Hefner dated 5/28/15.

LEGEND

Qaf	Quartnary Artificial Fill
Ta	Ardath Shale

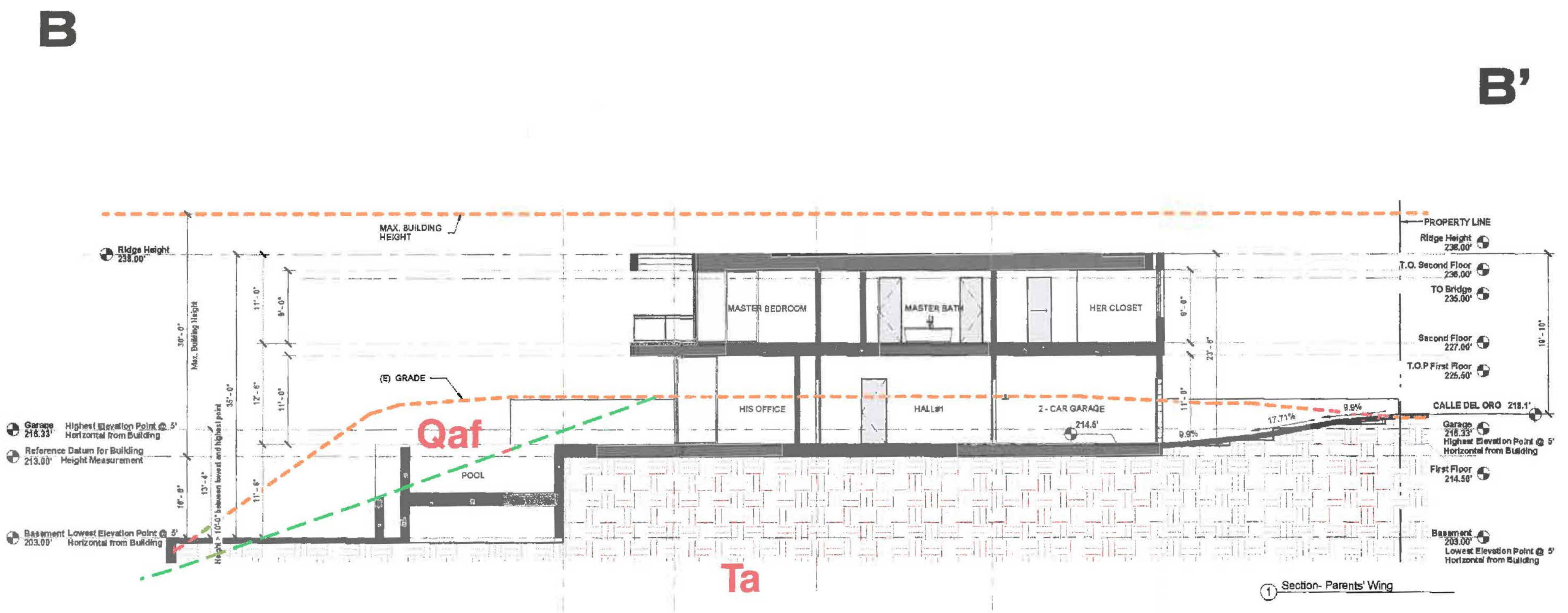
CROSS SECTION

Proposed Klein Residence
2585 Calle Del Oro
La Jolla, CA.
Figure No. VIa
Job No. 13-10407

Geotechnical Exploration, Inc.

July 2016

CROSS SECTION
B-B’



Scale: 1” = 20’
(approximate)

REFERENCE: This Cross Section was prepared from an existing SECTION PLANS by Studio William Hefner dated 5/28/15.

LEGEND

Qaf	Quartnary Artificial Fill
Ta	Ardath Shale

CROSS SECTION

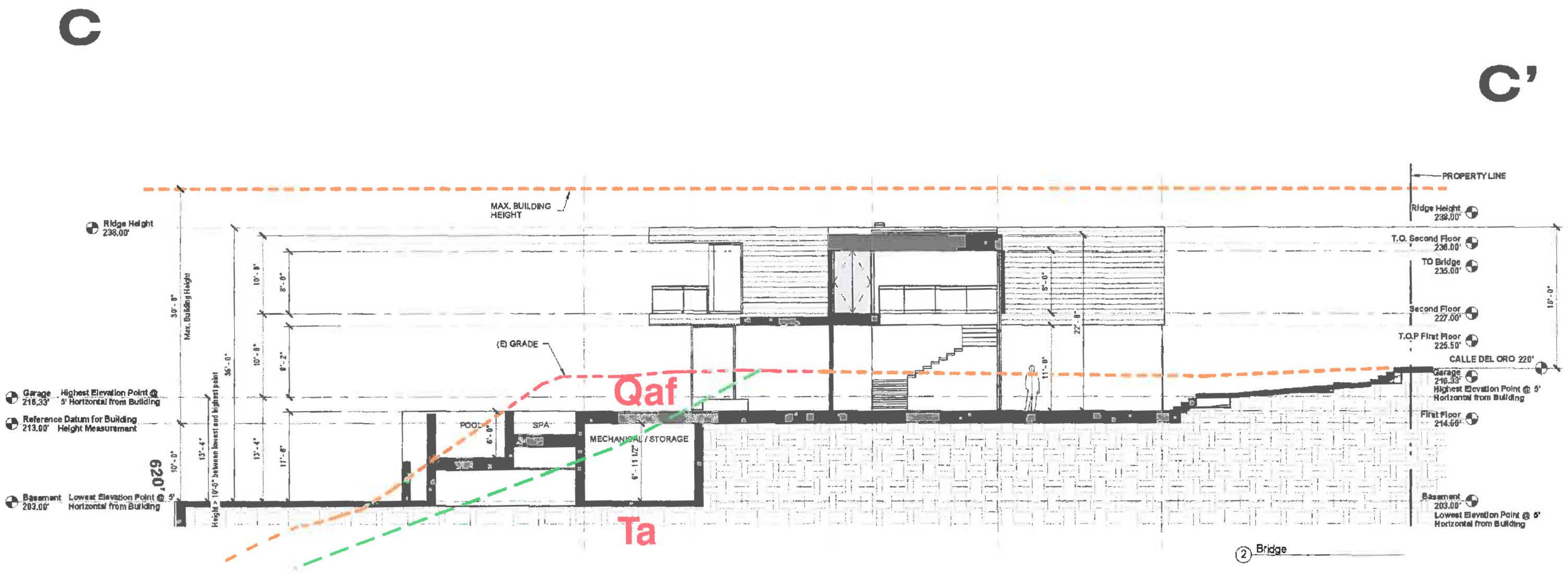
Proposed Klein Residence
2585 Calle Del Oro
La Jolla, CA.
Figure No. VIb
Job No. 13-10407

Geotechnical Exploration, Inc.
July 2016

13-10407-BB.ai

CROSS SECTION

C-C'



Scale: 1" = 20'
(approximate)

LEGEND

Qaf	Quartnary Artificial Fill
Ta	Ardath Shale

REFERENCE: This Cross Section was prepared from an existing SECTION PLANS by Studio William Hefner dated 5/28/15.

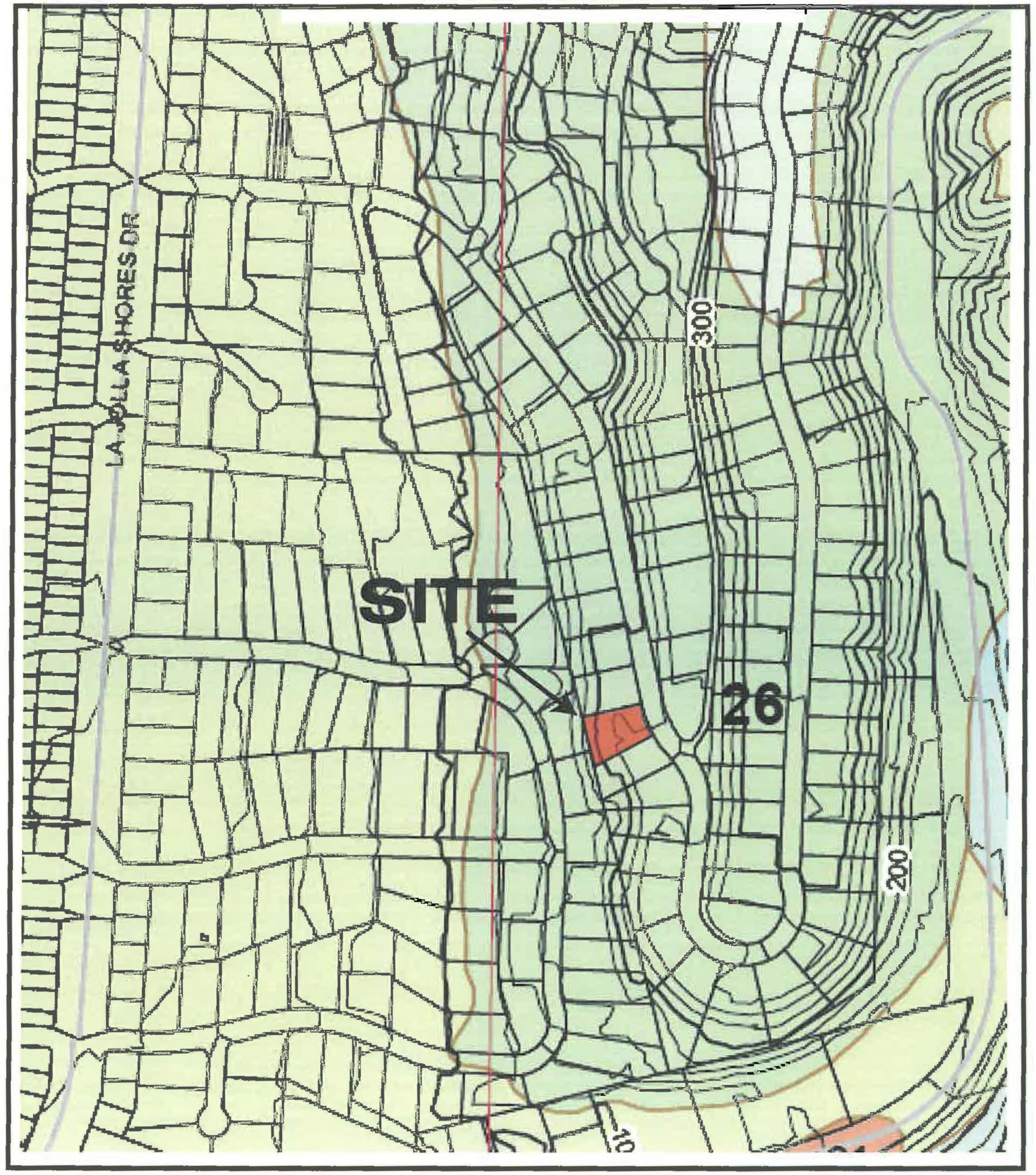
CROSS SECTION

Proposed Klein Residence
2585 Calle Del Oro
La Jolla, CA.
Figure No. VIc
Job No. 13-10407

Geotechnical Exploration, Inc.
July 2016

Geologic Hazards Map Excerpt
from City of San Diego
Geologic Hazards and Fault Map
Sheet 29

Development Services Department
DATE: 4/3/2008



Klein Residence Additions
2585 Calle Del Oro
La Jolla, CA.

LEGEND

Geologic Hazard Categories

FAULT ZONES

- ▨ 11 Active, Alquist-Priolo Earthquake Fault Zone
- ☰ 12 Potentially Active, Inactive, Presumed Inactive, or Activity Unknown
- 🟪 13 Downtown special fault zone

LANDSLIDES

- 🟥 21 Confirmed, known, or highly suspected
- 🟪 22 Possible or conjectured

SLIDE-PRONE FORMATIONS

- 🟪 23 Friars: neutral or favorable geologic structure
- 🟦 24 Friars: unfavorable geologic structure
- 🟦 25 Ardath: neutral or favorable geologic structure
- 🟩 26 Ardath: unfavorable geologic structure
- 🟪 27 Otay, Sweetwater, and others

LIQUEFACTION

- 🟨 31 High Potential -- shallow groundwater major drainages, hydraulic fills
- 🟨 32 Low Potential -- fluctuating groundwater minor drainages

COASTAL BLUFFS

- 🟧 41 Generally unstable Numerous landslides, high steep bluffs, severe erosion, unfavorable geologic structure
- 🟧 42 Generally unstable Unfavorable bedding plains, high erosion
- 🟧 43 Generally unstable Unfavorable jointing, local high erosion
- 🟧 44 Moderately stable Mostly stable formations, local high erosion
- 🟫 45 Moderately stable Some minor landslides, minor erosion
- 🟫 46 Moderately stable Some unfavorable geologic structure, minor or no erosion
- 🟧 47 Generally stable Favorable geologic structure, minor or no erosion, no landslides
- 🟨 48 Generally stable Broad beach areas, developed harbor

OTHER TERRAIN

- 🟩 51 Level mesas -- underlain by terrace deposits and bedrock nominal risk
- 🟩 52 Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk
- 🟩 53 Level or sloping terrain, unfavorable geologic structure, Low to moderate risk
- 🟩 54 Steeply sloping terrain, unfavorable or fault controlled geologic structure, Moderate risk
- 🟦 55 Modified terrain (graded sites) Nominal risk

Water (Bays and Lakes)

- 🟦

FAULTS

- ∿ Fault
- ∿ Inferred Fault
- Concealed Fault
- Shear Zone

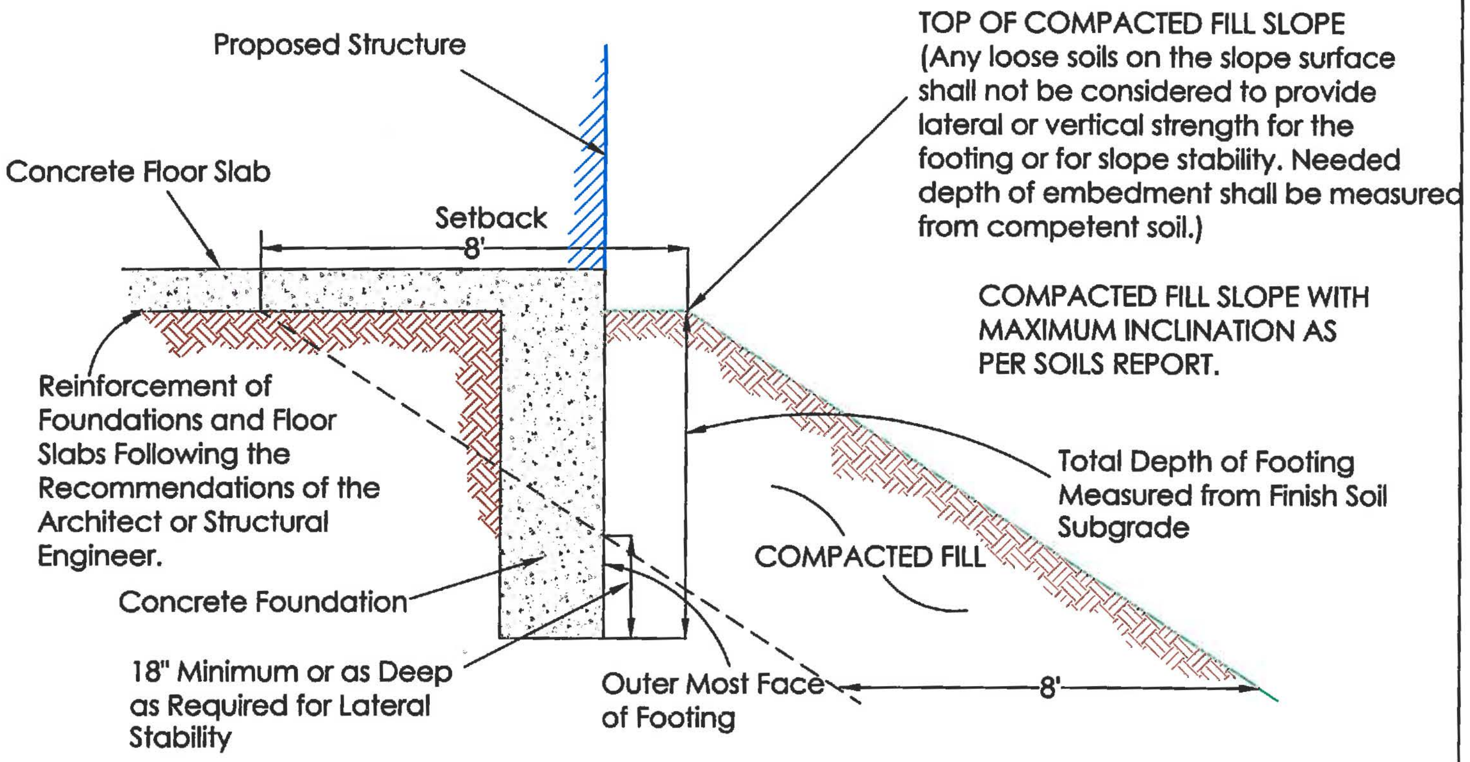
Figure No. VII
Job No. 13-10407

Geotechnical Exploration, Inc.

July 2016

klein-seis-combo.ai

FOUNDATION REQUIREMENTS NEAR SLOPES



TYPICAL SECTION

(Showing Proposed Foundation Located Within 8 Feet of Top of Slope)

18" FOOTING / 8' SETBACK

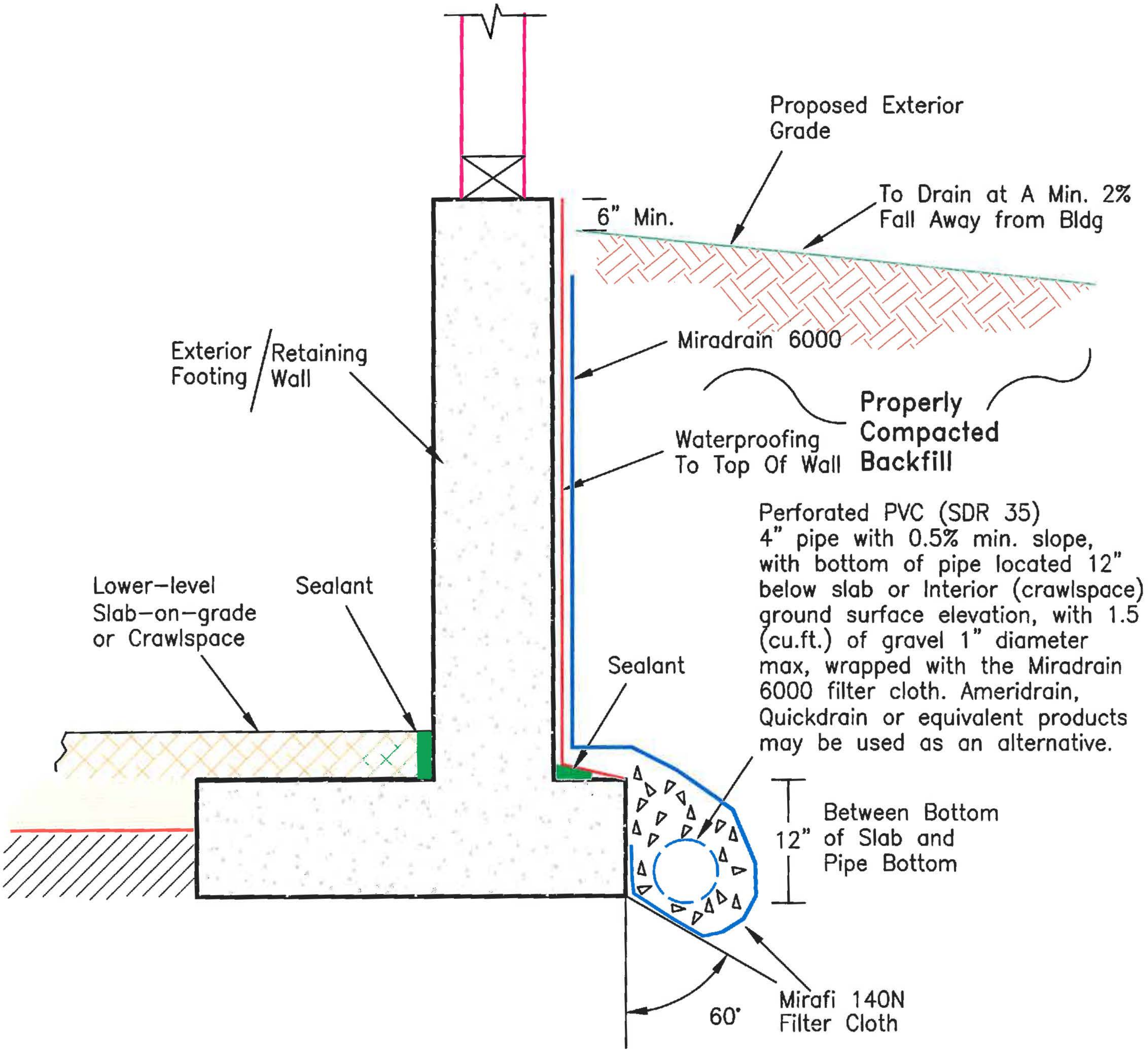
	Total Depth of Footing	
Distance From Top of Slope	1.5:1.0 SLOPE *	2.0:1.0 SLOPE
0	82"	66"
2'	66"	54"
4'	51"	42"
6'	34"	30"
8'	18"	18"

* when applicable

Figure No. VIII
Job No. 13-10407

Geotechnical Exploration, Inc.

RECOMMENDED SUBGRADE RETAINING WALL DRAINAGE SCHEMATIC



NOT TO SCALE

Figure No. IX

Job No. 13-10407

NOTE: As an option to Miradrain 6000, Gravel or Crushed rock 3/4" maximum diameter may be used with a minimum 12" thickness along the interior face of the wall and 2.0 cu.ft./ft. of pipe gravel envelope.

Geotechnical Exploration, Inc.

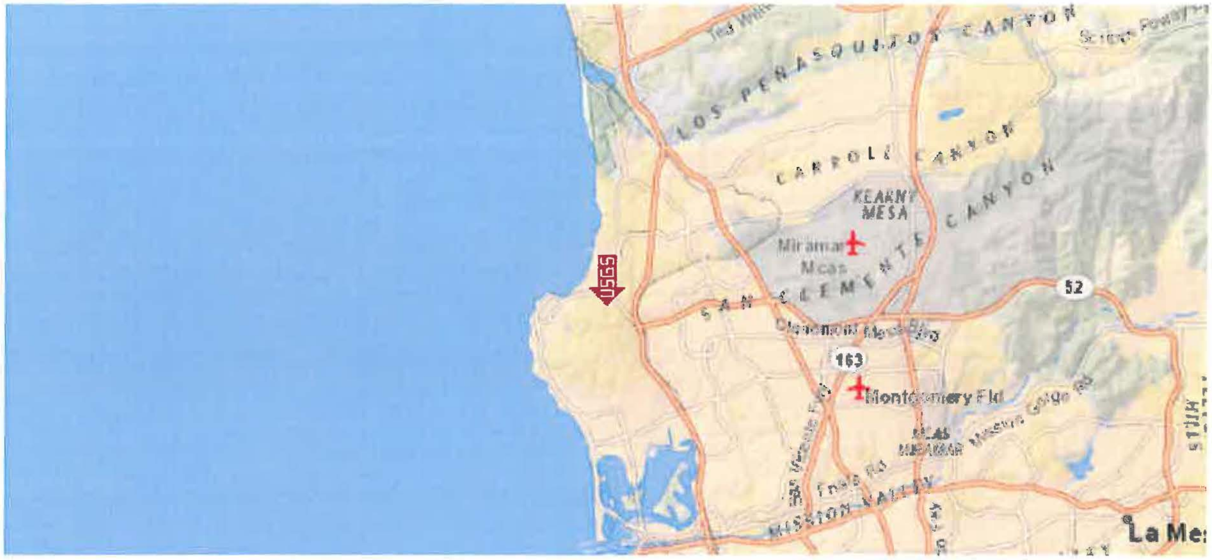
APPENDIX B

USGS DESIGN MAPS SUMMARY REPORT

USGS Design Maps Summary Report

User–Specified Input

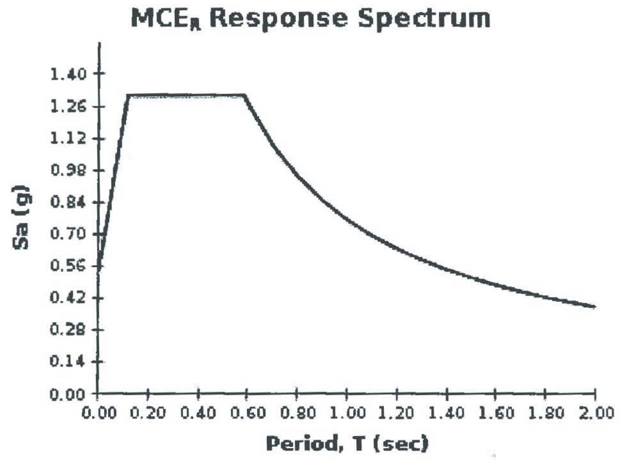
Report Title	Klein Residence Mon September 19, 2016 22:34:33 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	32.855°N, 117.2489°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

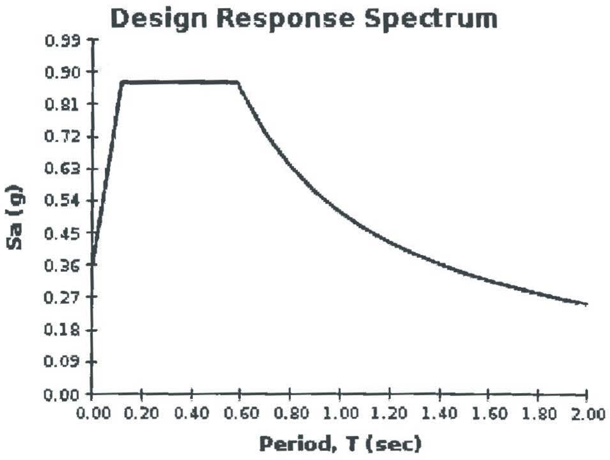


USGS–Provided Output

$S_S =$ 1.307 g	$S_{MS} =$ 1.307 g	$S_{DS} =$ 0.871 g
$S_1 =$ 0.508 g	$S_{M1} =$ 0.762 g	$S_{D1} =$ 0.508 g

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

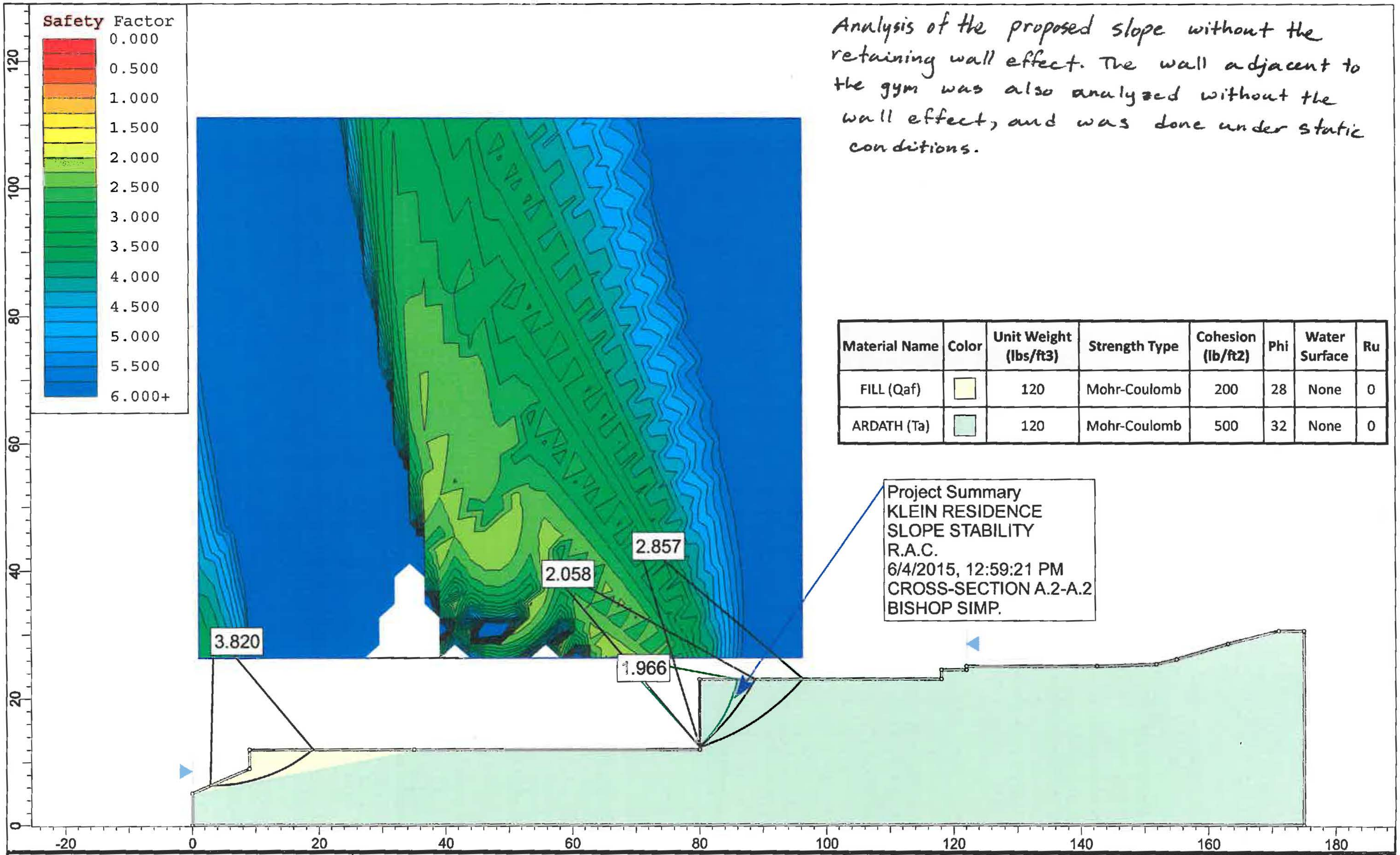




For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

APPENDIX C

SLOPE STABILITY CALCULATIONS



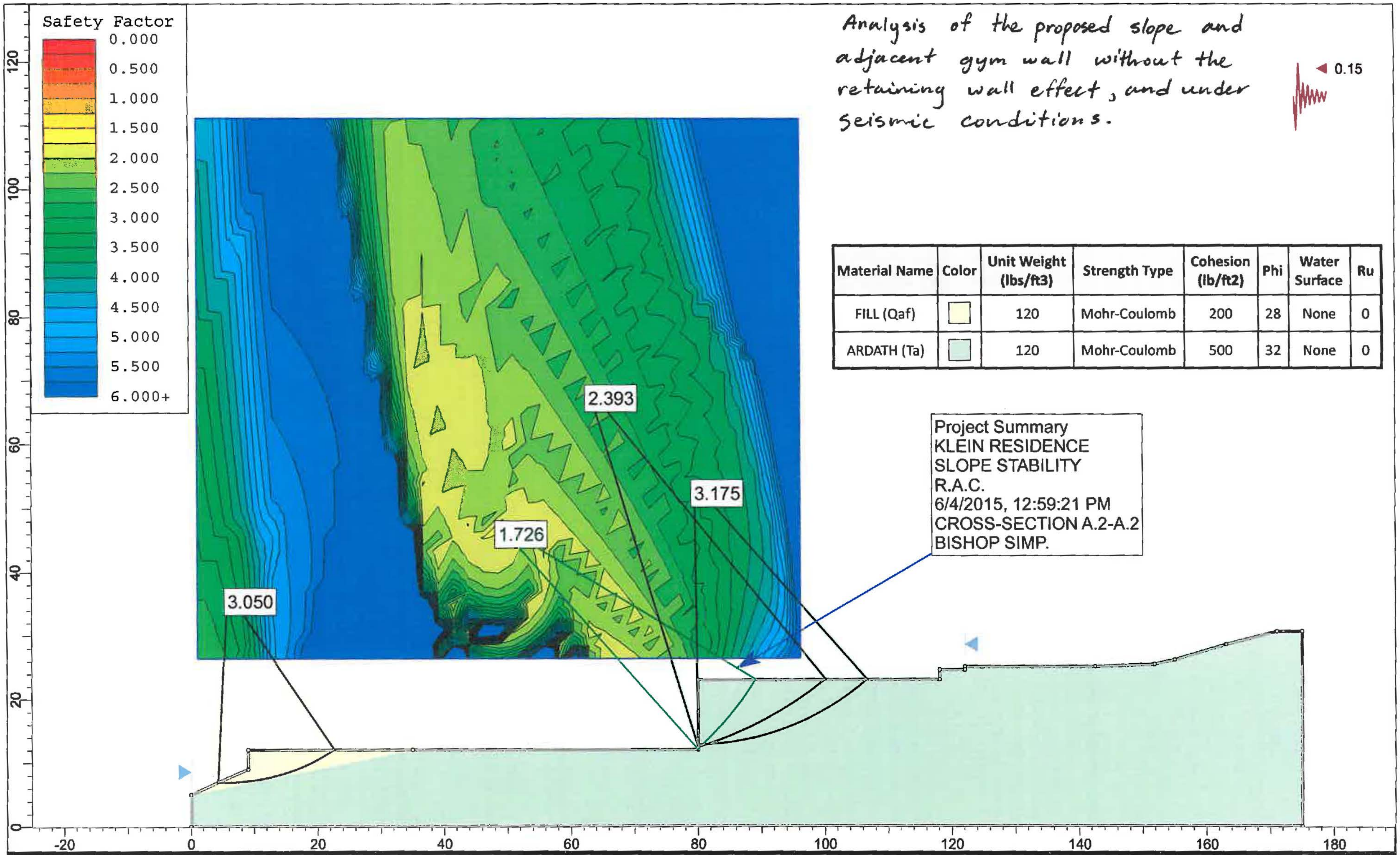
Analysis of the proposed slope without the retaining wall effect. The wall adjacent to the gym was also analyzed without the wall effect, and was done under static conditions.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/4/2015, 12:59:21 PM
CROSS-SECTION A.2-A.2
BISHOP SIMP.

SLIDEINTERPRET 6.005

Project	KLEIN RESIDENCE
Analysis Description	SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:250
Company	
Date	6/4/2015, 12:59:21 PM
File Name	JOB NO.13-10407A.201.slim



Analysis of the proposed slope and adjacent gym wall without the retaining wall effect, and under seismic conditions.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

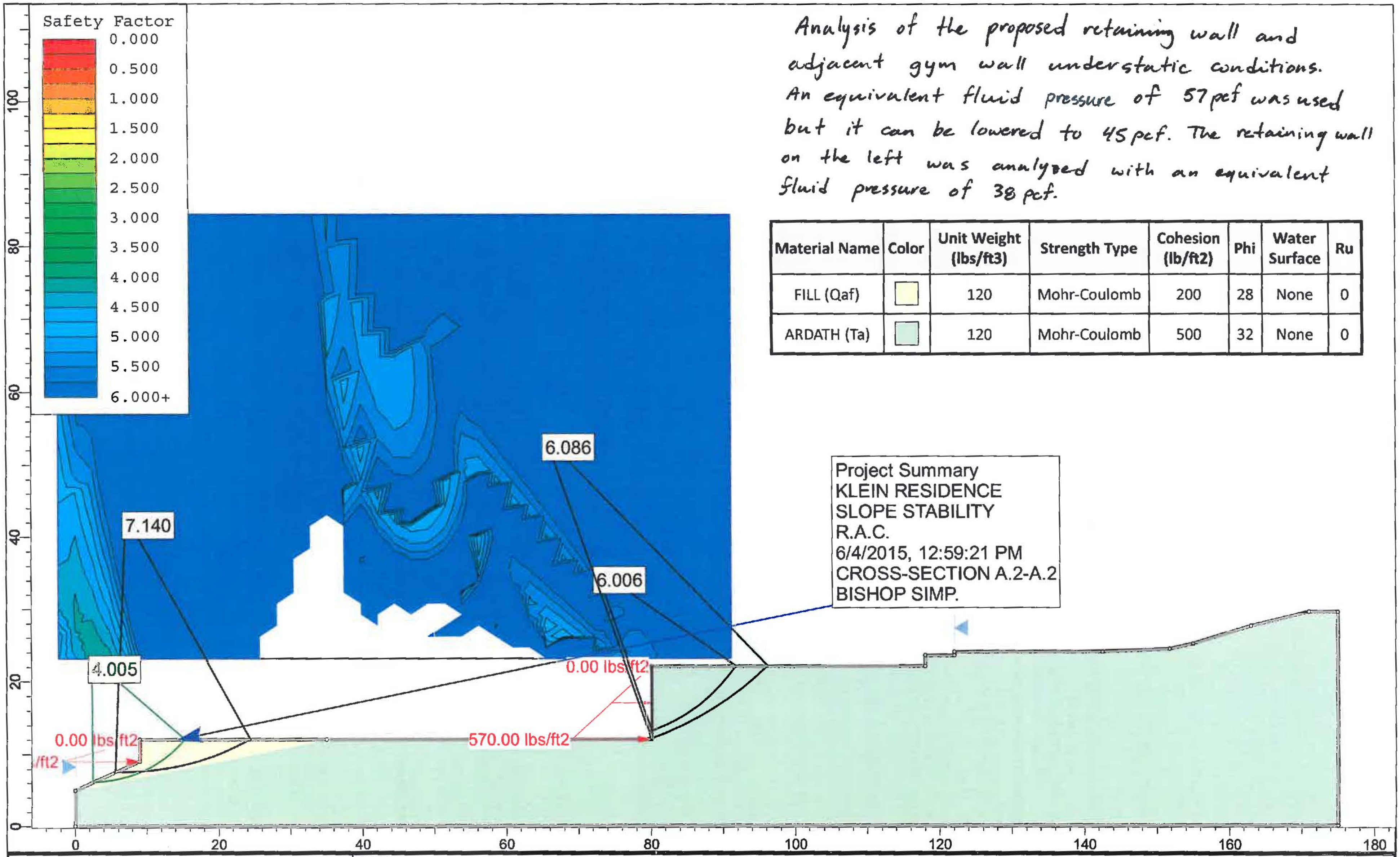
Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/4/2015, 12:59:21 PM
CROSS-SECTION A.2-A.2
BISHOP SIMP.

SLIDEINTERPRET 6.005

Project	KLEIN RESIDENCE
Analysis Description	SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:250
Company	
Date	6/4/2015, 12:59:21 PM
File Name	JOB NO.13-10407A.201w_0.15gSHAKE.slim

Analysis of the proposed retaining wall and adjacent gym wall understatic conditions. An equivalent fluid pressure of 57 pcf was used but it can be lowered to 45 pcf. The retaining wall on the left was analyzed with an equivalent fluid pressure of 38 pcf.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0



Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/4/2015, 12:59:21 PM
CROSS-SECTION A.2-A.2
BISHOP SIMP.

SLIDEINTERPRET 6.005

Project: KLEIN RESIDENCE

Analysis Description: SLOPE STABILITY

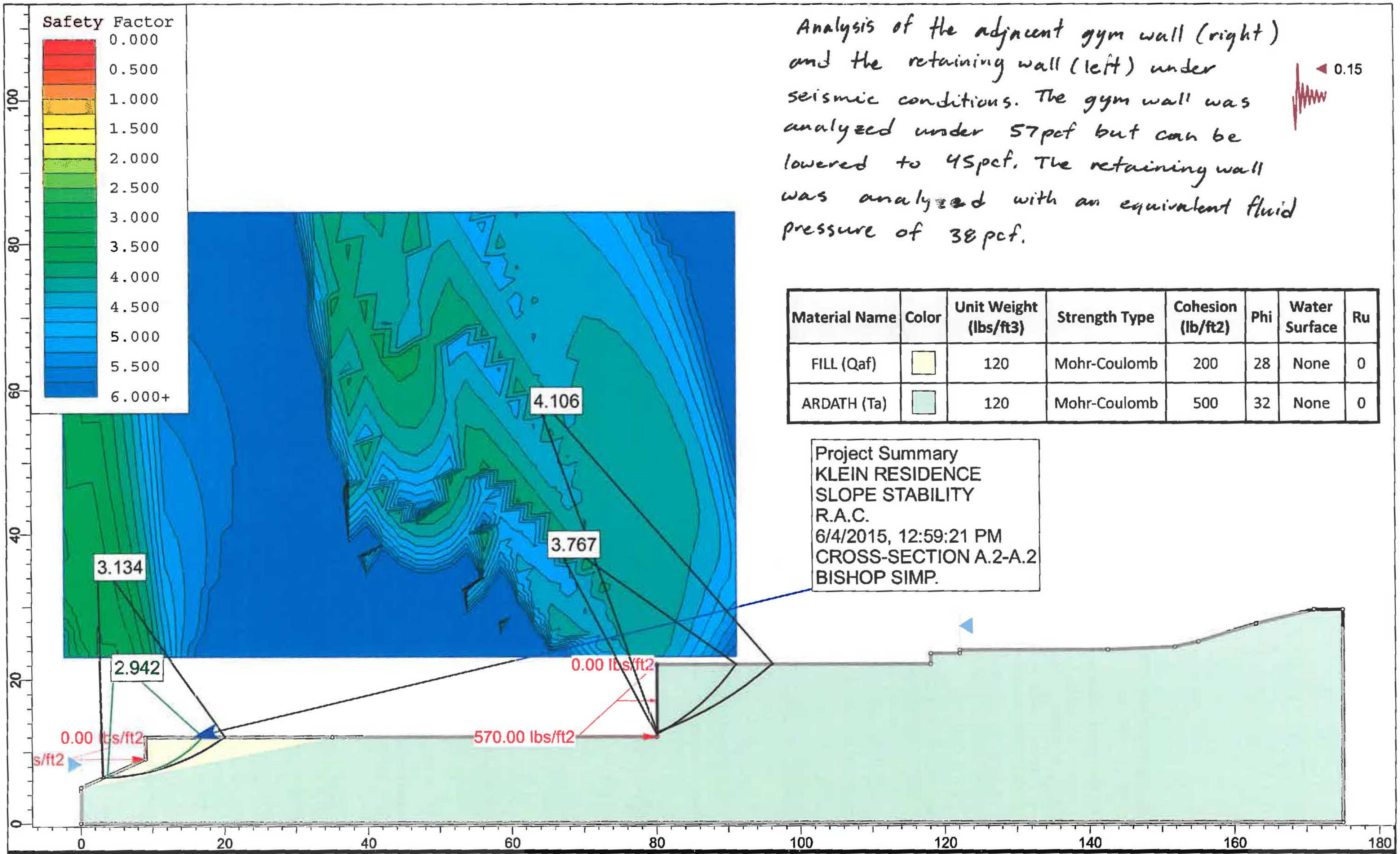
Drawn By: R.A.C.

Scale: 1:220

Company:

Date: 6/4/2015, 12:59:21 PM

File Name: JOB NO.13-10407A.202.slim



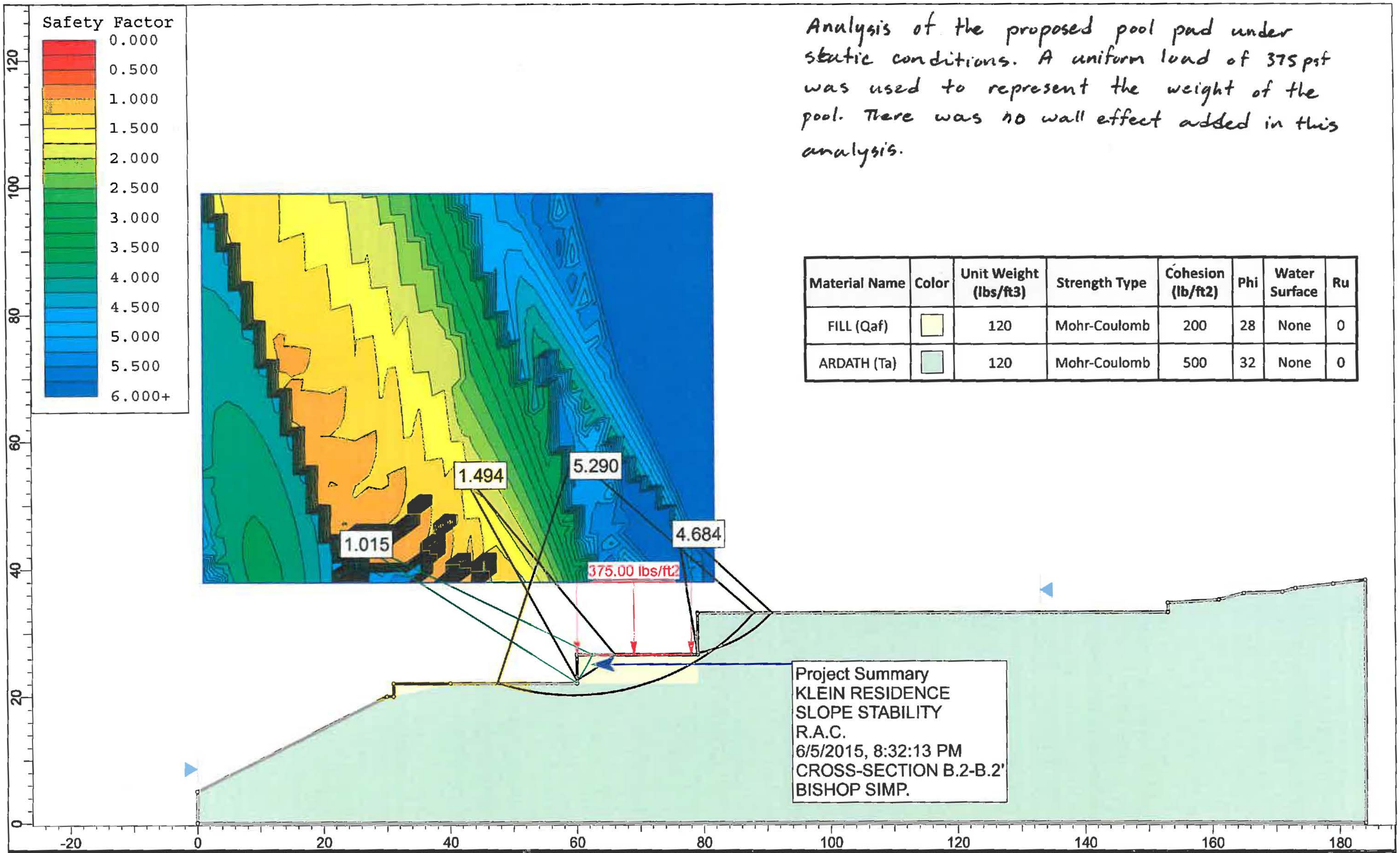
Analysis of the adjacent gym wall (right) and the retaining wall (left) under seismic conditions. The gym wall was analyzed under 57pcf but can be lowered to 45pcf. The retaining wall was analyzed with an equivalent fluid pressure of 38pcf.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/4/2015, 12:59:21 PM
CROSS-SECTION A.2-A.2
BISHOP SIMP.

Project	KLEIN RESIDENCE
Analysis Description	SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:220
Company	
Date	6/4/2015, 12:59:21 PM
File Name	JOB NO.13-10407A.202w_0.15gSHAKE.slim

SLIDEINTERPRET 6.005



Analysis of the proposed pool pad under static conditions. A uniform load of 375 psf was used to represent the weight of the pool. There was no wall effect added in this analysis.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/5/2015, 8:32:13 PM
CROSS-SECTION B.2-B.2'
BISHOP SIMP.

SLIDEINTERPRET 6.005

Project
KLEIN RESIDENCE

Analysis Description
SLOPE STABILITY

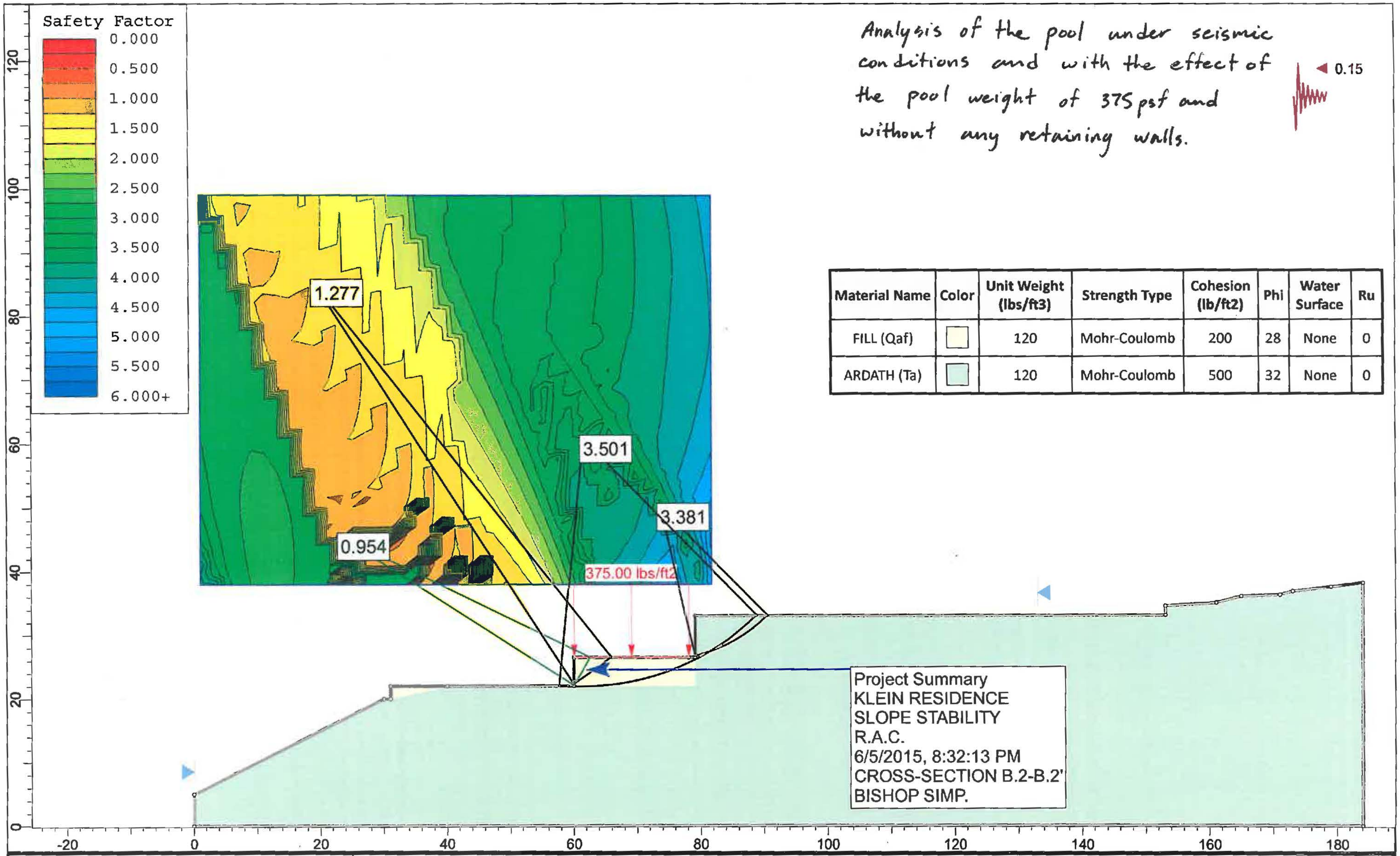
Drawn By R.A.C.

Scale 1:250

Company

Date 6/5/2015, 8:32:13 PM

File Name JOB NO. 13-10407B.201.slim



Analysis of the pool under seismic conditions and with the effect of the pool weight of 375 psf and without any retaining walls.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/5/2015, 8:32:13 PM
CROSS-SECTION B.2-B.2'
BISHOP SIMP.

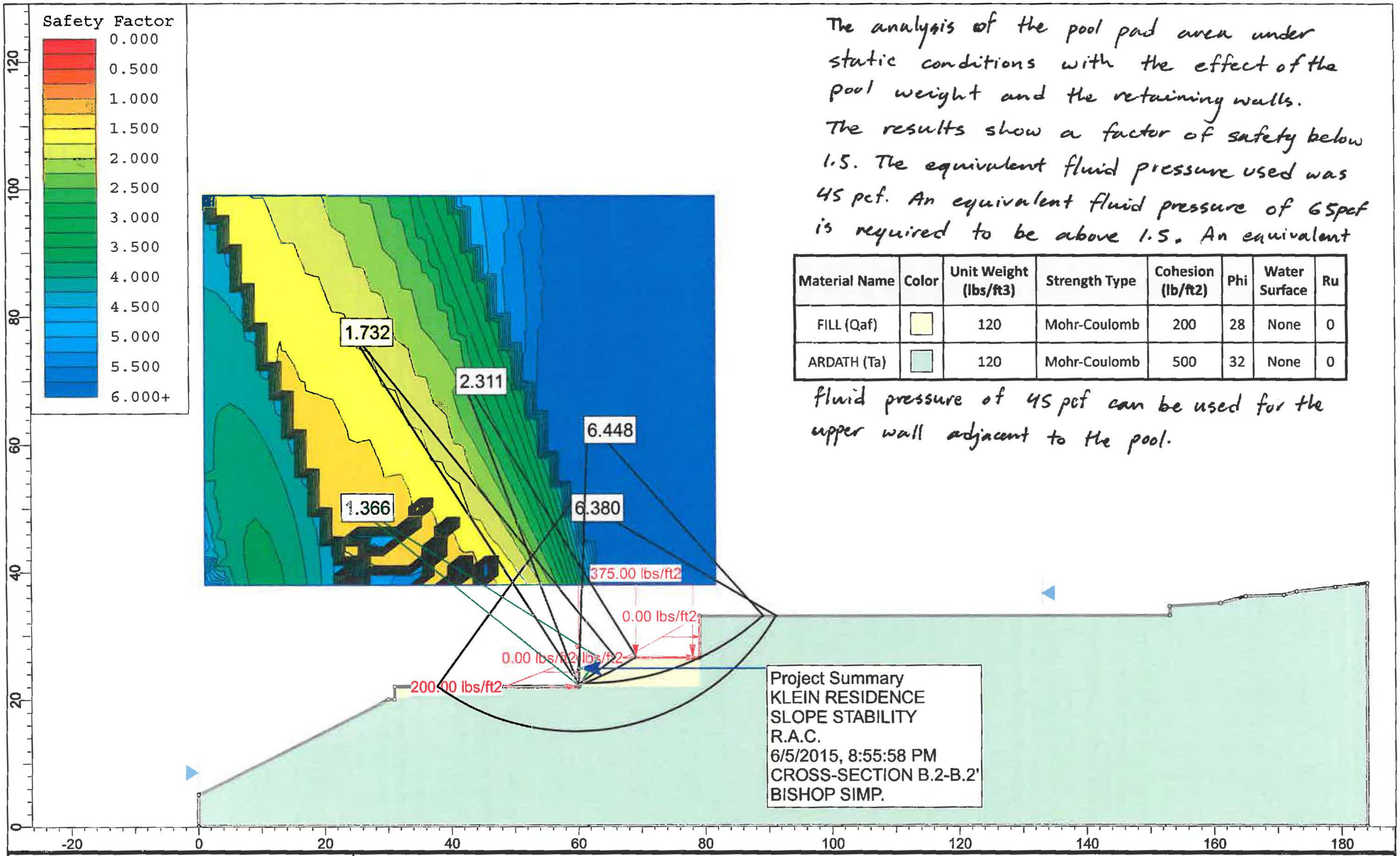
SLIDEINTERPRET 6.005

Project KLEIN RESIDENCE

Analysis Description SLOPE STABILITY

Drawn By R.A.C. *Scale* 1:250 *Company*

Date 6/5/2015, 8:32:13 PM *File Name* JOB NO. 13-10407B.201w_0.15gSHAKE.slim



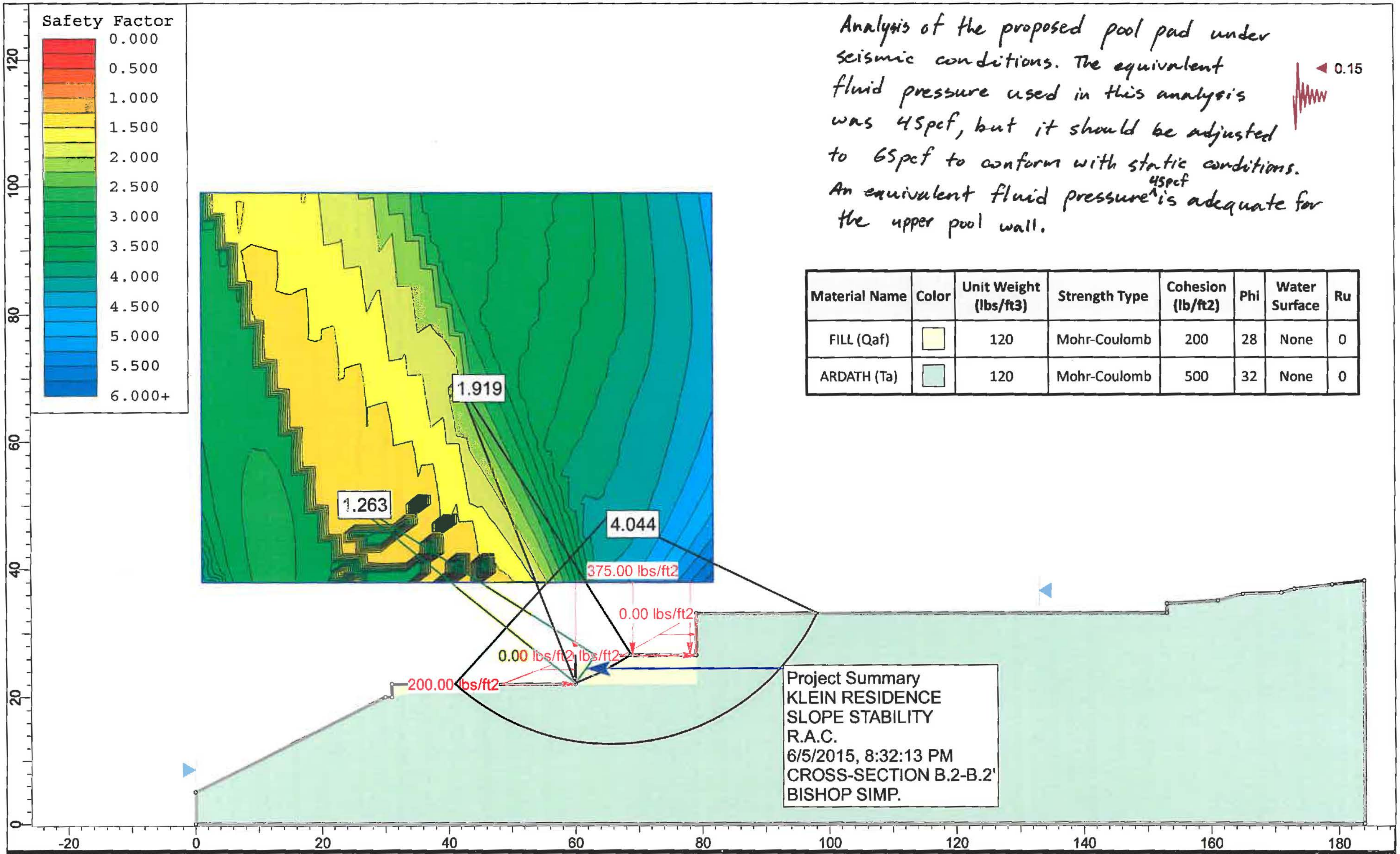
The analysis of the pool pad area under static conditions with the effect of the pool weight and the retaining walls. The results show a factor of safety below 1.5. The equivalent fluid pressure used was 45 pcf. An equivalent fluid pressure of 65pcf is required to be above 1.5. An equivalent fluid pressure of 45 pcf can be used for the upper wall adjacent to the pool.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/5/2015, 8:55:58 PM
CROSS-SECTION B.2-B.2'
BISHOP SIMP.

SLIDEINTERPRET 6.005

Project	KLEIN RESIDENCE		
Analysis Description	SLOPE STABILITY		
Drawn By	R.A.C.	Scale	1:250
Company			
Date	6/5/2015, 8:55:58 PM	File Name	JOB NO. 13-10407B.202.slim



Analysis of the proposed pool pad under seismic conditions. The equivalent fluid pressure used in this analysis was 45pcf, but it should be adjusted to 65pcf to conform with static conditions. An equivalent fluid pressure 45pcf is adequate for the upper pool wall.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

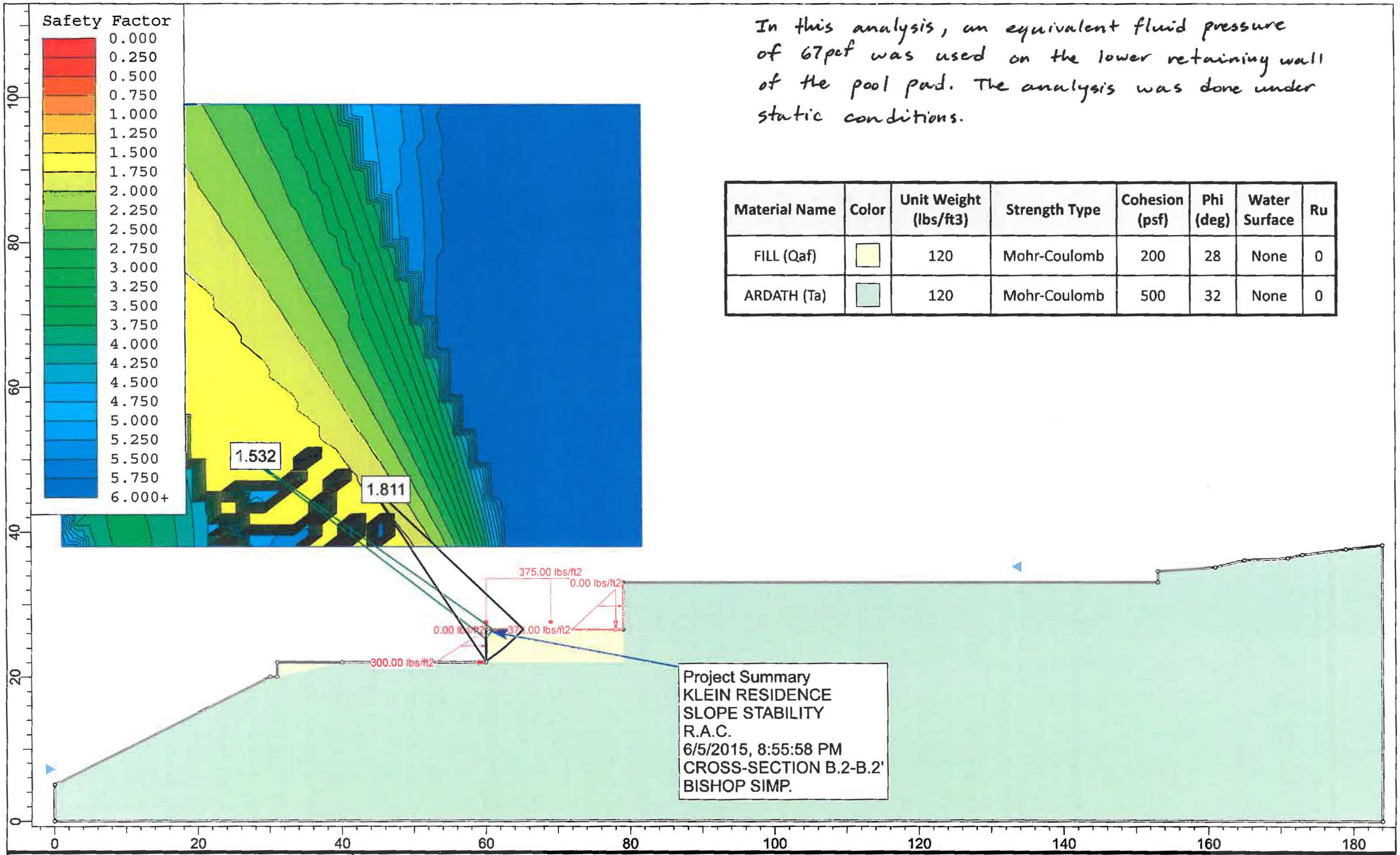
Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/5/2015, 8:32:13 PM
CROSS-SECTION B.2-B.2'
BISHOP SIMP.

SLIDEINTERPRET 6.005

Project	KLEIN RESIDENCE		
Analysis Description	SLOPE STABILITY		
Drawn By	R.A.C.	Scale	1:250
Company			
Date	6/5/2015, 8:32:13 PM	File Name	JOB NO. 13-10407B.202w_0.15gSHAKE.slim

In this analysis, an equivalent fluid pressure of 67pcf was used on the lower retaining wall of the pool pad. The analysis was done under static conditions.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

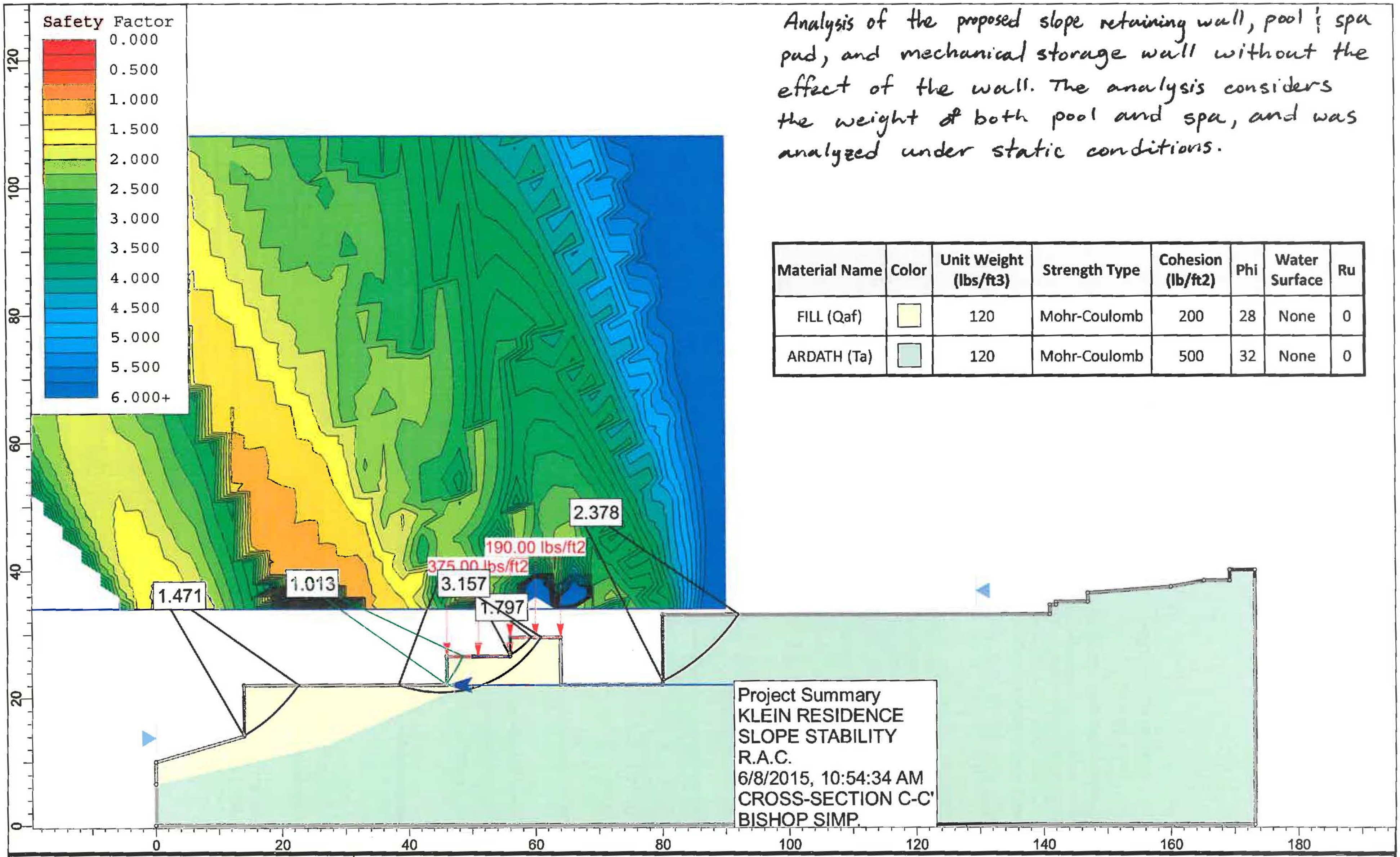


Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/5/2015, 8:55:58 PM
CROSS-SECTION B.2-B.2'
BISHOP SIMP.

Project	KLEIN RESIDENCE
Analysis Description	SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:220
Company	
Date	6/5/2015, 8:55:58 PM
File Name	JOB NO. 13-10407B.203.slim

rocscience

SLIDEINTERPRET 6.035



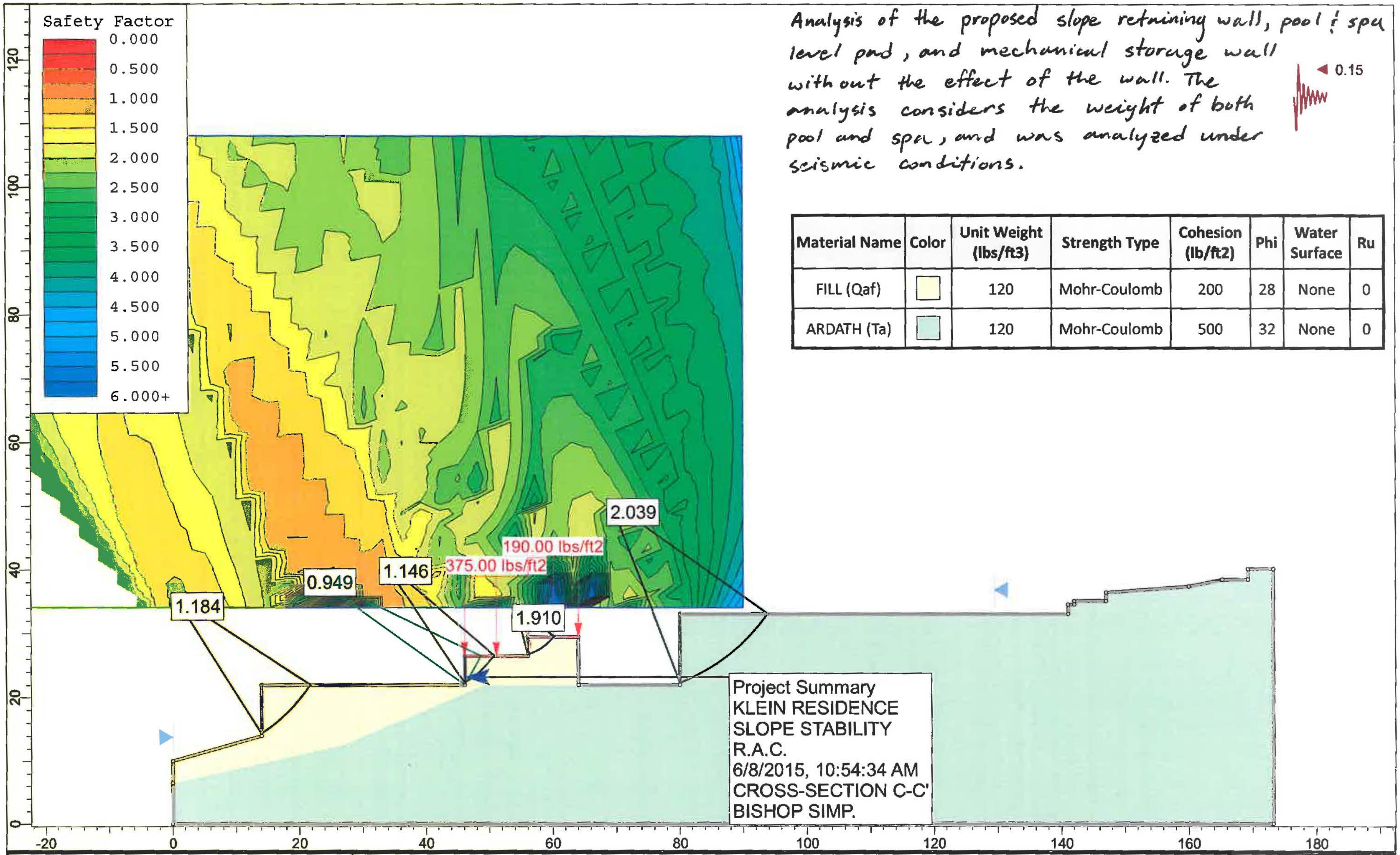
Analysis of the proposed slope retaining wall, pool & spa pad, and mechanical storage wall without the effect of the wall. The analysis considers the weight of both pool and spa, and was analyzed under static conditions.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/8/2015, 10:54:34 AM
CROSS-SECTION C-C'
BISHOP SIMP.

Project	KLEIN RESIDENCE		
Analysis Description	SLOPE STABILITY		
Drawn By	R.A.C.	Scale	1:250
Company			
Date	6/8/2015, 10:54:34 AM	File Name	JOB NO. 13-10407C01.slim

SLIDEINTERPRET 6.005



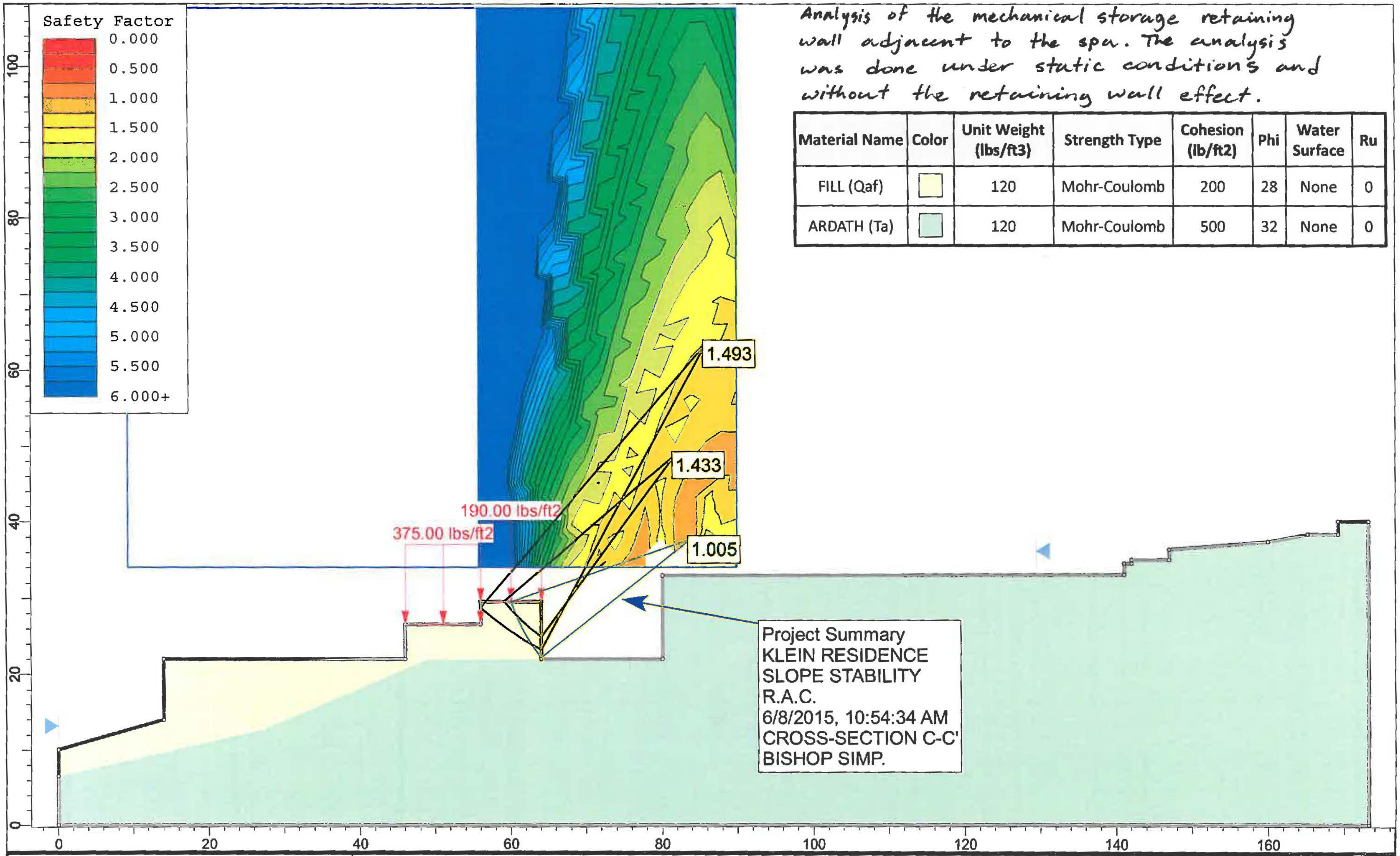
Analysis of the proposed slope retaining wall, pool & spa level pad, and mechanical storage wall without the effect of the wall. The analysis considers the weight of both pool and spa, and was analyzed under seismic conditions.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟦	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/8/2015, 10:54:34 AM
CROSS-SECTION C-C'
BISHOP SIMP.

SLIDEINTERPRET 6.005

Project: KLEIN RESIDENCE
Analysis Description: SLOPE STABILITY
Drawn By: R.A.C.
Scale: 1:250
Company:
Date: 6/8/2015, 10:54:34 AM
File Name: JOB NO. 13-10407C01w_0.15gSHAKE.slim



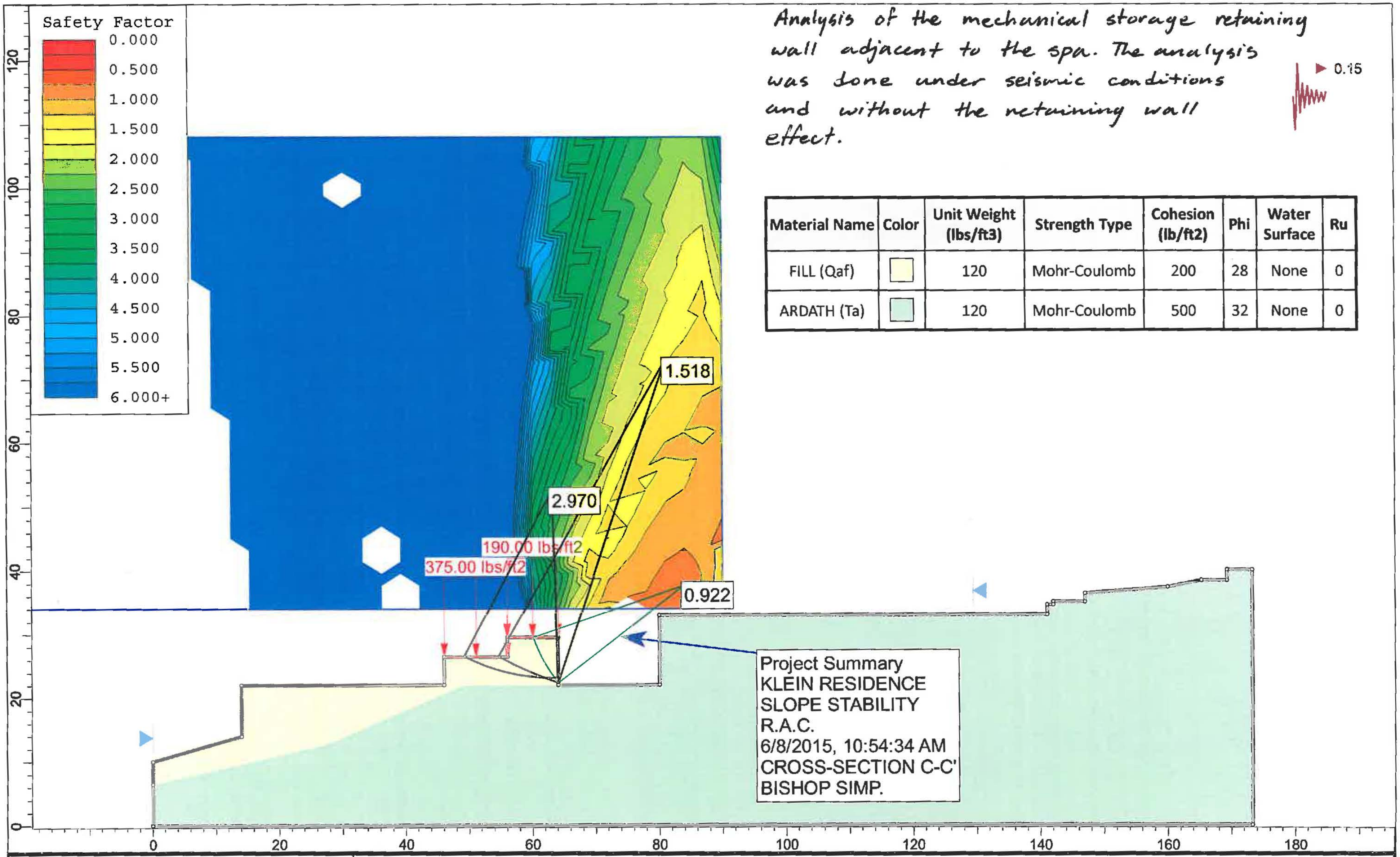
Analysis of the mechanical storage retaining wall adjacent to the spa. The analysis was done under static conditions and without the retaining wall effect.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/8/2015, 10:54:34 AM
CROSS-SECTION C-C'
BISHOP SIMP.

SLIDEINTERPRET 6.005

Project	KLEIN RESIDENCE		
Analysis Description	SLOPE STABILITY		
Drawn By	R.A.C.	Scale	1:210
Company			
Date	6/8/2015, 10:54:34 AM	File Name	JOB NO. 13-10407C02.slim



Analysis of the mechanical storage retaining wall adjacent to the spa. The analysis was done under seismic conditions and without the retaining wall effect.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/8/2015, 10:54:34 AM
CROSS-SECTION C-C'
BISHOP SIMP.

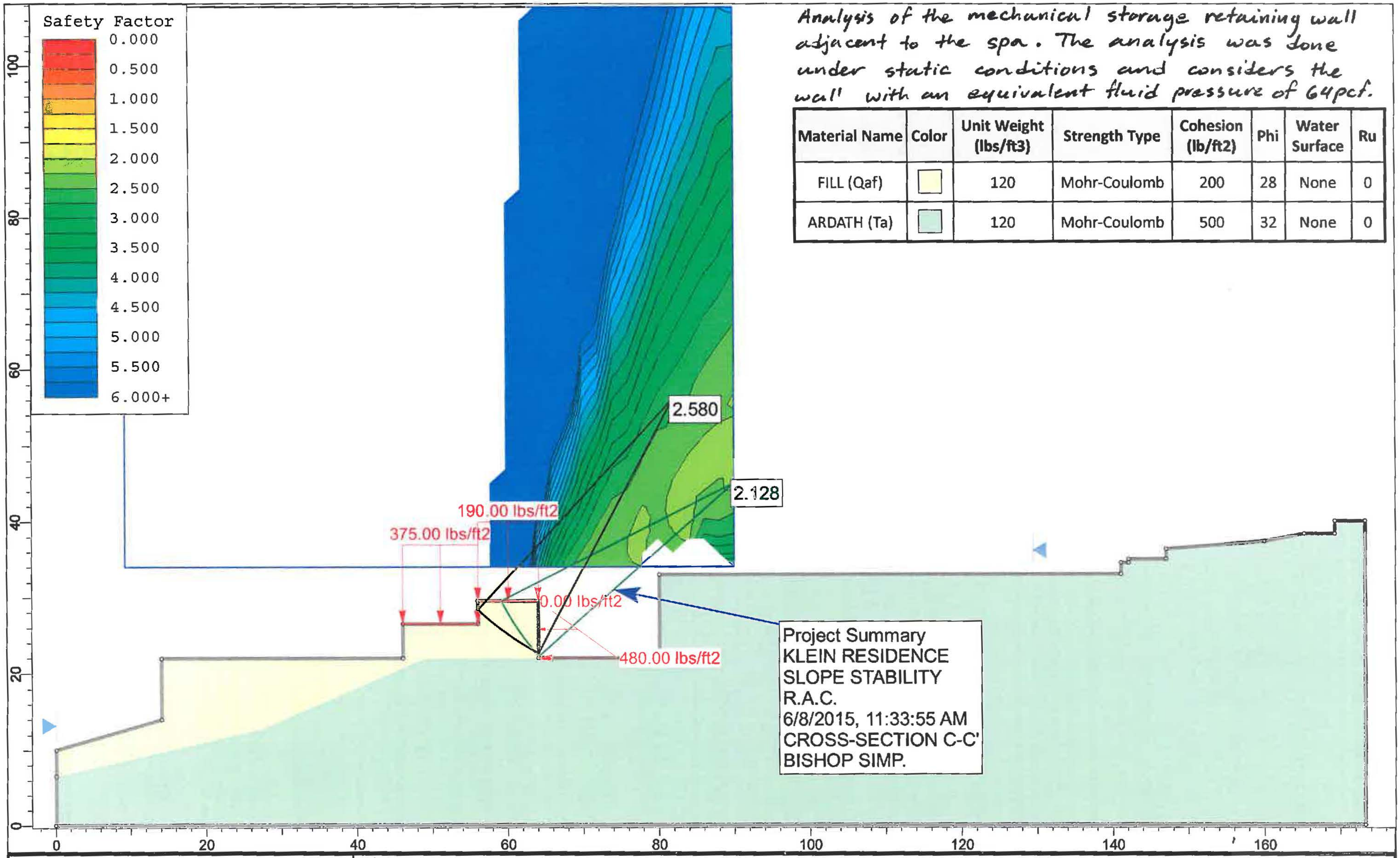
SLIDEINTERPRET 6.005

Project
KLEIN RESIDENCE

Analysis Description
SLOPE STABILITY

Drawn By R.A.C. *Scale* 1:250 *Company*

Date 6/8/2015, 10:54:34 AM *File Name* JOB NO. 13-10407C02w_0.15gSHAKE.slim



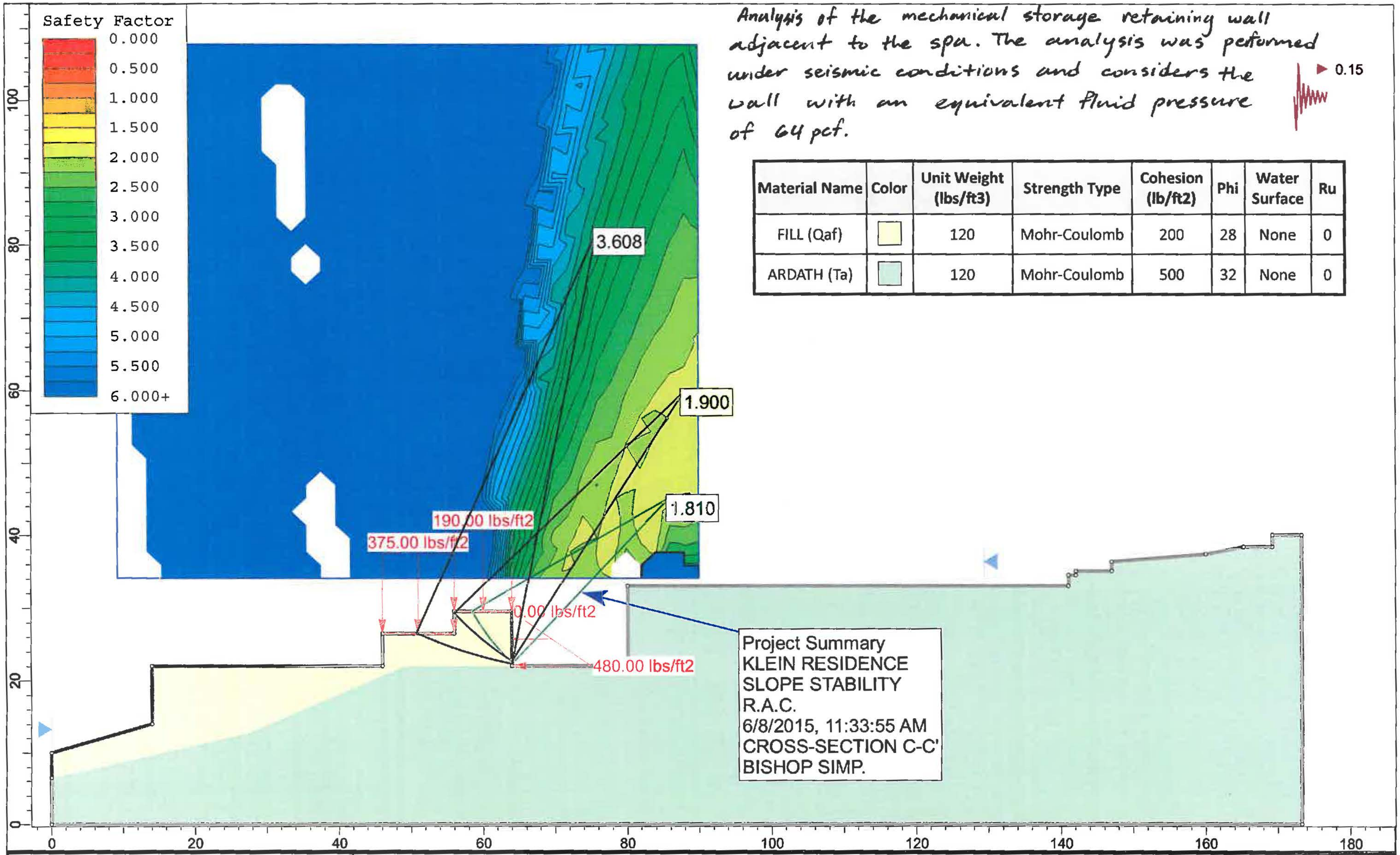
Analysis of the mechanical storage retaining wall adjacent to the spa. The analysis was done under static conditions and considers the wall with an equivalent fluid pressure of 64pcf.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/8/2015, 11:33:55 AM
CROSS-SECTION C-C'
BISHOP SIMP.

Project	KLEIN RESIDENCE
Analysis Description	SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:210
Company	
Date	6/8/2015, 11:33:55 AM
File Name	JOB NO. 13-10407C03.slim

SLIDEINTERPRET 6.005



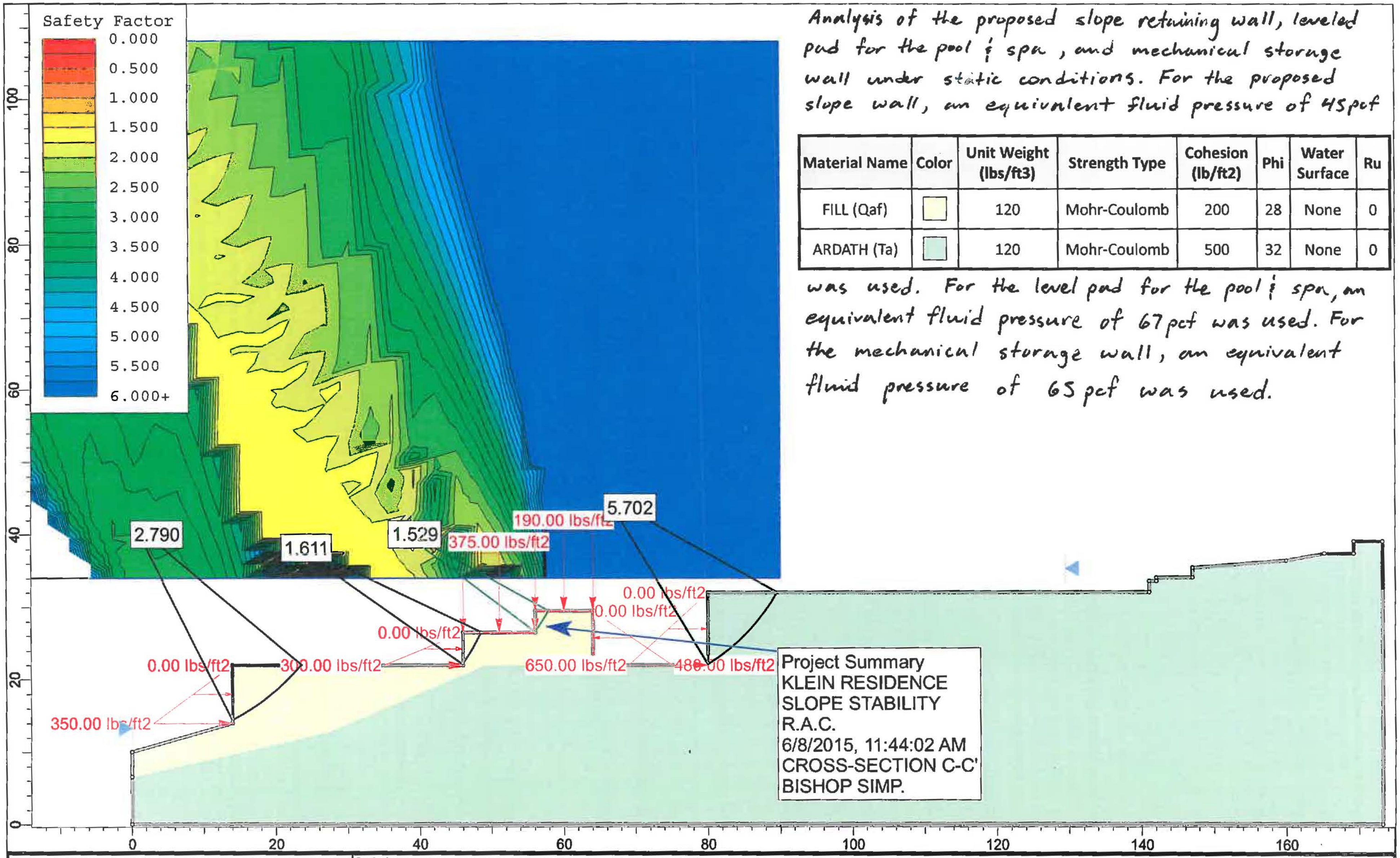
Analysis of the mechanical storage retaining wall adjacent to the spa. The analysis was performed under seismic conditions and considers the wall with an equivalent fluid pressure of 64 pcf.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/8/2015, 11:33:55 AM
CROSS-SECTION C-C'
BISHOP SIMP.

Project	KLEIN RESIDENCE
Analysis Description	SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:220
Company	
Date	6/8/2015, 11:33:55 AM
File Name	JOB NO. 13-10407C03w_0.15gSHAKE.slim

SLIDEINTERPRET 6.005



Analysis of the proposed slope retaining wall, leveled pad for the pool & spa, and mechanical storage wall under static conditions. For the proposed slope wall, an equivalent fluid pressure of 45pcf was used. For the level pad for the pool & spa, an equivalent fluid pressure of 67 pcf was used. For the mechanical storage wall, an equivalent fluid pressure of 65 pcf was used.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/8/2015, 11:44:02 AM
CROSS-SECTION C-C'
BISHOP SIMP.

SLIDEINTERPRET 6.005

Project	KLEIN RESIDENCE
Analysis Description	SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:220
Company	
Date	6/8/2015, 11:44:02 AM
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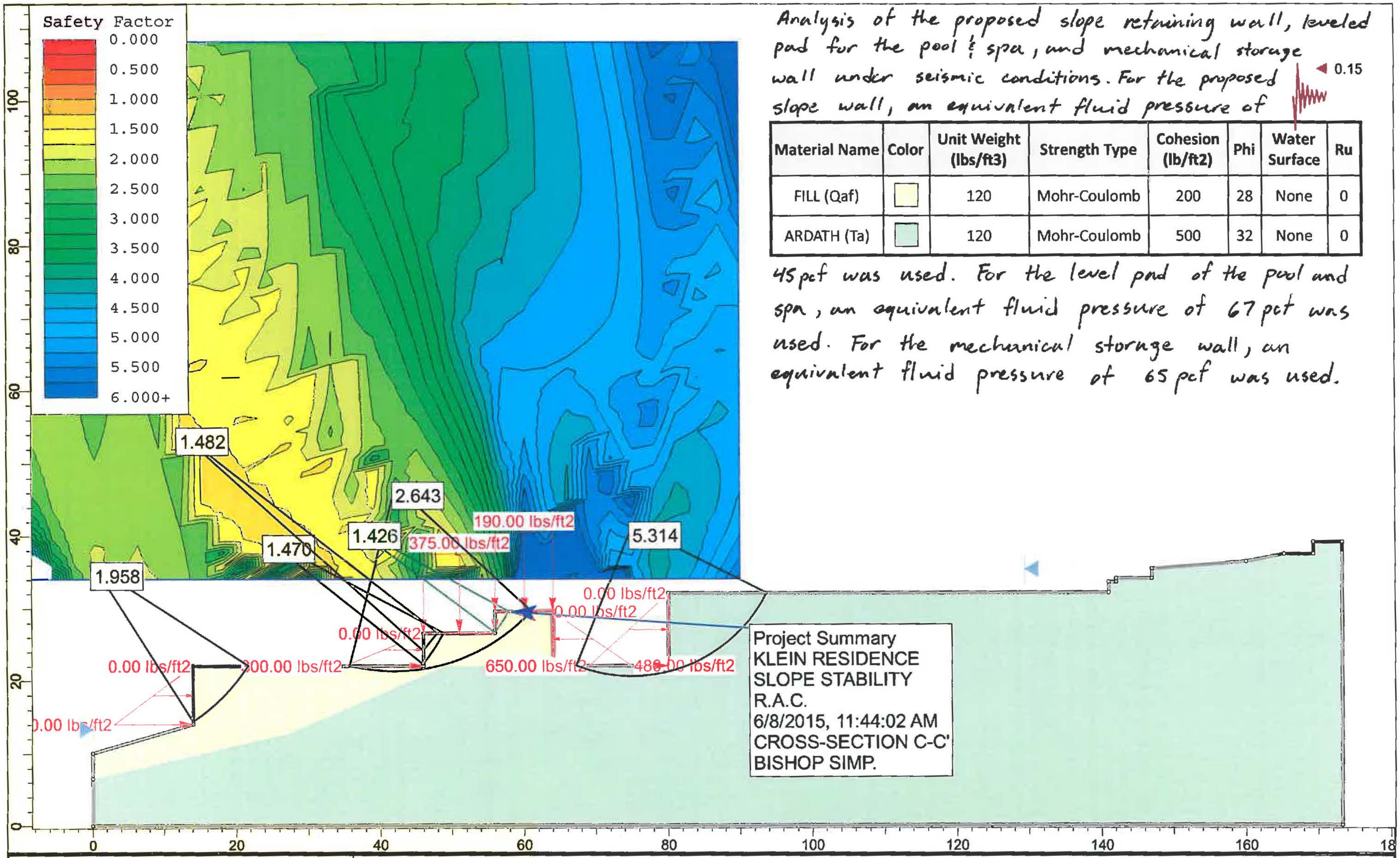

Analysis of the proposed slope retaining wall, leveled pad for the pool & spa, and mechanical storage wall under seismic conditions. For the proposed slope wall, an equivalent fluid pressure of 45 pcf was used. For the level pad of the pool and spa, an equivalent fluid pressure of 67 pcf was used. For the mechanical storage wall, an equivalent fluid pressure of 65 pcf was used.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Water Surface	Ru
FILL (Qaf)	🟨	120	Mohr-Coulomb	200	28	None	0
ARDATH (Ta)	🟩	120	Mohr-Coulomb	500	32	None	0

Project Summary
KLEIN RESIDENCE
SLOPE STABILITY
R.A.C.
6/8/2015, 11:44:02 AM
CROSS-SECTION C-C'
BISHOP SIMP.

SLIDEINTERPRET 6.005

Project: KLEIN RESIDENCE
Analysis Description: SLOPE STABILITY
Drawn By: R.A.C.
Scale: 1:220
Company:
Date: 6/8/2015, 11:44:02 AM
File Name: JOB NO. 13-10407C04w_0.15gSHAKE.slim

Shallow Failure Analysis
Slope Stability Calculations

Klein Property
2585 Calle del Oro
La Jolla, California

Job No. 13-10407

Soil Design Parameters

Soil Unit Weight: 120 pcf; Saturated Unit Weight: 130 pcf
Friction Angle: 28 degrees in fill, 32 degrees in formation
Cohesion: 200 psf in fill, 500 psf in formation
Slope Angle, β: 26.56 degrees (existing steep slope in formational soils)
and 34.8 degrees (existing slope in fill soils)

Shallow Failure Stability Analysis

In fill soils:

$Fs= C/(\gamma \text{ sat. } H. \cos^{\wedge}2\ (\beta).\ \text{Tan } \beta) \quad + \quad (\gamma'/\gamma \text{ sat})(\tan \phi/\tan\beta)$

$= 200/(130 \times 3.0 \times 0.673 \times 0.695) \quad + \quad (67.6/130)\ (0.532/0.695)$

$= 1.096 + 0.398$

$= 1.495$ rounded to 1.5 **OK**

In Formational Soils:

$Fs= 500/(130 \times 3 \times 0.800 \times 0.500) + (67.6/130)\ (0.625/0.500)$

$= 3.205 + 0.65= 3.86 > 1.5$ **O.K**.

APPENDIX D

SIMPLE OPEN PIT TEST RESULTS AND INFILTRATION RATE CONVERSIONS

Percolation Test Sheet

Project Name: Klein
Calculated By: JAB
Date: 7/19/16
Soil Classification: (CL)
Project No. 13-10407
Checked By:
Date:
Date Excavated: 7/18/16
Test Hole No: INF-1
Test Hole Dia: 24"
Depth of Test Hole: 37"

	Time (minutes)	Time interval	Initial water level	Final water level (inches)	Change in water (inches)	Percolation rate (min/inches)
	212	20	30.750	32.875	2.125	9.412
	232					
	232	20	32.875	34.250	1.375	14.545
	252					
	252	20	34.250	35.250	1.000	20.000
	312					
	317	60	30.500	33.750	3.250	18.462
	417					
	419	60	30.500	33.500	3.000	20.000
	519					

Simple Open Pit Rate to Infiltration Rate Conversion (Porchet Method)

Project Name: Klein	Calculated By: JAB	Date: 7/19/16
Project No. 13-10407	Checked By:	Date:
Test Hole No: INF-1	Test Hole Dia: 24"	Depth of Test Hole: 37"

Porchet Corrections

Infiltration rate=((delta h*60r)/(delta t*(r+2 h avg))

Test No.	Delta T (min)	h 1 (inches)	h 2 (inches)	delta h (inches)	h avg (inches)	r (radius) (inches)	delta h*60r	delta t*(r+2 h	Infiltration rate (in/hr)
1	20	6.250	4.125	2.125	5.188	12	1530	447.5	3.419
2	20	4.125	2.750	1.375	3.438	12	990	377.5	2.623
3	20	2.750	1.750	1.000	2.250	12	720	330	2.182
4	60	6.500	3.250	3.250	4.875	12	2340	1305	1.793
5	60	6.500	3.500	3.000	5.000	12	2160	1320	1.636
6									
7									
8									
9									

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.

Questions pertaining to the Checklist should be directed to Development Services Department at 619-446-5000.

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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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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 3. 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 that would result in an increase in GHG emissions when compared to the existing designations, is nevertheless consistent with the assumptions in the CAP because it would implement CAP Strategy 3 actions. 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)

Table 4 **Size-based Trigger Levels for Electric Vehicle Charging Requirements for Non-Residential Buildings related to Question 10: Electric Vehicle Charging supporting Strategy 3: Bicycling, Walking, Transit & Land Use of the Climate Action Plan**

Land Use Type	Size-based Trigger Level
Hospital	500 or more beds OR Expansion of a 500+ bed hospital by 20%
College	3,000 or more students OR Expansion of a 3,000+ student college by 20%
Hotels/Motels	500 or more rooms
Industrial, Manufacturing or Processing Plants or Industrial Parks	1,000 or more employees OR 40 acres or more of land area OR 650,000 square feet or more of gross floor area
Office buildings or Office Parks	1,000 or more employees OR 250,000 square feet or more of gross floor area
Shopping centers or Trade Centers	1,000 or more employees OR 500,000 square feet or more of gross floor area
Sports, Entertainment or Recreation Facilities	Accommodate at least 4,000 persons per performance OR Contain 1,500 or more fixed seats
Transit Projects (including, but not limited to, transit stations and park and ride lots).	All

Source: Adapted from the Governor's Office of Planning and Research's (OPR's) Model Building Code for Plug-In Electric Vehicle Charging

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Worksheet C.4-1: Categorization of Infiltration Feasibility Condition

<table>
<tr><th colspan="2">Categorization of Infiltration Feasibility Condition</th><th colspan="2">Worksheet C.4-1</th></tr>
<tr><td colspan="4">Part 1 - Full Infiltration Feasibility Screening Criteria
Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?</td></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>1</td><td>Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
The infiltration test results below the proposed facility location was 0.818 inches per hour with a minimum factor of safety of 2 applied. Simple open pit testing was performed at 1 locations on the site within the proposed infiltration basins in accordance with Appendix D of the City of San Diego BMP design manual. In addition, a comprehensive evaluation of the site was conducted in accordance with Appendix C.2. Please refer to our "Addendum Geotechnical Report" dated August 4, 2016 for details of the comprehensive evaluation and investigation conducted, simple open pit test results and converted simple open pit test results to infiltration rate calculations, and maps representative of the study.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>2</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:
The infiltration test results below the proposed facility locations was 0.818 inches per hour with a minimum factor of safety of 2 applied. In our opinion, any long term infiltration at the site will result in geotechnical hazards which cannot be reasonably mitigated to an acceptable level. Based on our comprehensive evaluation in accordance with Appendix C.2, the anticipated geotechnical hazards are outlined below:
C.2.3 Slope Stability
The formational soils underlying the site consist of sandy silts and sandy clays of the Ardath Shale Formation. The City of San Diego Geologic Hazards Map Sheet 29 indicates the site is located in geologic hazard category 26 "Ardath: unfavorable geologic structure". Any attempt at infiltration on the site may increase the risk of slope failure on the westerly descending slope, potentially damaging property and life on the site and adjacent properties.
C.2.4 Utility Considerations
Water intrusion into existing utility lines and vaults both on-site and off-site including the adjacent public streets (Calle Del Oro) and existing development which bounds the western and northern property boundary at a lower elevation.
C.2.6 Retaining Walls and Foundations
Water migration under proposed building foundations and proposed retaining walls resulting in increased lateral pressures and reduction in soil strength.
Please refer to our "Addendum Geotechnical Report" dated August 4, 2016 for details.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
</table>

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 2 of 4</th></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:
In review of our "Report of Limited Geotechnical Investigation" dated February 10, 2014, groundwater was not encountered to a depth of 11.5 feet below existing ground surface. Although groundwater was not encountered to the aforementioned depth, the risk for groundwater related concerns include shallow perched seepage due to the practically impermeable, hard, nature of the silty and clayey formational soils across the site. Based on our comprehensive evaluation in accordance with Appendix C.3, the "other factors" associated with the risk of groundwater contamination include shallow perched seepage that cannot be mitigated to an acceptable level. The anticipated risk for groundwater related concerns are outlined below:
C.3.7 Other Factors
Inadequate infiltration treatment of shallow perched seepage surfacing on the adjacent slopes to the north and south potentially transporting storm water pollutants or other factors.
Please refer to our "Addendum Geotechnical Report" dated August 4, 2016 for details of the comprehensive evaluation and investigation conducted, simple open pit test results and converted simple open pit test results to infiltration rate calculations, and maps representative of the study.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td></td></tr>
<tr><td colspan="4">Provide basis:
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result*</td><td colspan="2">If all answers to rows 1 - 4 are "Yes" a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration
If any answer from row 1-4 is "No", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2</td><td></td></tr>
</table>

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

Worksheet C.4-1 Page 3 of 4

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria
Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X

Provide basis:

Measured infiltration rate on the site was 0.818 inches per hour with a minimum factor of safety of 2 applied. Although the measured infiltration rate is considered to be appreciable, the geologic conditions on the site are considered unfavorable due to the silty and clayey nature of the formational soils increasing the potential for slope instability. In addition, the fractured nature of the Ardath Shale underlying the proposed infiltration basin allowed for higher than normal infiltration rates typical of this silty/clayey formation.
Please refer to our "Addendum Geotechnical Report" dated August 4, 2016 for details of the comprehensive evaluation and investigation conducted, simple open pit test results and converted simple open pit test results to infiltration rate calculations, and maps representative of the study.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Criteria	Screening Question	Yes	No
6	Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X

Provide basis:

In our opinion, any long term infiltration at the site will result in geotechnical hazards which cannot be reasonably mitigated to an acceptable level. Based on our comprehensive evaluation in accordance with Appendix C.2, the anticipated geotechnical hazards are outlined below:
C.2.3 Slope Stability
The formational soils underlying the site consist of sandy silts and sandy clays of the Ardath Shale Formation. The City of San Diego Geologic Hazards Map Sheet 29 indicates the site is located in geologic hazard category 26 "Ardath: unfavorable geologic structure". Any attempt at infiltration on the site may increase the risk of slope failure on the westerly descending slope, potentially damaging property and life on the site and adjacent properties.
C.2.4 Utility Considerations
Water intrusion into existing utility lines and vaults both on-site and off-site including the adjacent public streets (Calle Del Oro) and existing development which bounds the western and northern property boundary at a lower elevation.
C.2.6 Retaining Walls and Foundations
Water migration under proposed building foundations and proposed retaining walls resulting in increased lateral pressures and reduction in soil strength. Please refer to our "Addendum Geotechnical Report" dated August 4, 2016 for details.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 4 of 4</th></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>7</td><td>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:
In review of our "Report of Limited Geotechnical Investigation" dated February 10, 2014, groundwater was not encountered to a depth of 11.5 feet below existing ground surface. Although groundwater was not encountered to the aforementioned depth, the risk for groundwater related concerns include shallow perched seepage due to the practically impermeable hard nature of the silty and clayey formational soils across the site. Based on our comprehensive evaluation in accordance with Appendix C.3, the "other factors" associated with the risk of groundwater contamination include shallow perched seepage that cannot be mitigated to an acceptable level. The anticipated risk for groundwater related concerns are outlined below:
C.3.7 Other Factors
Inadequate infiltration treatment of shallow perched seepage surfacing on the adjacent slopes to the north and south potentially transporting storm water pollutants or other factors.
Please refer to our "Addendum Geotechnical Report" dated August 4, 2016 for details of the comprehensive evaluation and investigation conducted, simple open pit test results and converted simple open pit test results to infiltration rate calculations, and maps representative of the study.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td></td></tr>
<tr><td colspan="4">Provide basis:
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.
If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td></td></tr>
</table>

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings

The City of San Diego

PRIORITY DEVELOPMENT PROJECT (PDP) STORM WATER QUALITY MANAGEMENT PLAN (SWQMP) FOR

Klein Residence
Permit Application Number: 441535
Drawing Number :__________

ENGINEER OF WORK:

REGISTERED PROFESSIONAL ENGINEER
PATRIC T. DE BOER
No. C 83583
EXP. 3/31/19
CIVIL
STATE OF CALIFORNIA

Patric de Boer

Patric T. de Boer RCE 83583
Provide Wet Signature and Stamp Above Line

PREPARED FOR:

The Trevor & Staci Klein Trust
2585 Calle Del Oro
La Jolla, CA 92037
858.459.1350

PREPARED BY:

Omega Engineering Consultants, Inc.
4340 Viewridge Ave, Suite B
San Diego, CA 92123
858.634.8620

DATE:

May 9, 2017

Approved by: City of San Diego Date

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Project Name: Klein Residence

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ACRONYMS

APN	Assessor's Parcel Number
ASBS	Area of Special Biological Significance
BMP	Best Management Practice
CEQA	California Environmental Quality Act
CGP	Construction General Permit
DCV	Design Capture Volume
DMA	Drainage Management Areas
ESA	Environmentally Sensitive Area
GLU	Geomorphic Landscape Unit
GW	Ground Water
HMP	Hydromodification Management Plan
HSG	Hydrologic Soil Group
HU	Harvest and Use
INF	Infiltration
LID	Low Impact Development
LUP	Linear Underground/Overhead Projects
MS4	Municipal Separate Storm Sewer System
N/A	Not Applicable
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
PDP	Priority Development Project
PE	Professional Engineer
POC	Pollutant of Concern
SC	Source Control
SD	Site Design
SDRWQCB	San Diego Regional Water Quality Control Board
SIC	Standard Industrial Classification
SWPPP	Stormwater Pollutant Protection Plan
SWQMP	Storm Water Quality Management Plan
TMDL	Total Maximum Daily Load
WMAA	Watershed Management Area Analysis
WPCP	Water Pollution Control Program
WQIP	Water Quality Improvement Plan

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CERTIFICATION PAGE

Project Name: Klein Residence
Permit Application Number: PTS 441535

I hereby declare that I am the Engineer in Responsible Charge of design of storm water BMPs for this project, and that I have exercised responsible charge over the design of the project as defined in Section 6703 of the Business and Professions Code, and that the design is consistent with the requirements of the Storm Water Standards, which is based on the requirements of SDRWQCB Order No. R9-2013-0001 as amended by R9-2015-0001 and R9-2015-0100 (MS4 Permit).

I have read and understand that the City Engineer has adopted minimum requirements for managing urban runoff, including storm water, from land development activities, as described in the Storm Water Standards. I certify that this PDP SWQMP has been completed to the best of my ability and accurately reflects the project being proposed and the applicable source control and site design BMPs proposed to minimize the potentially negative impacts of this project's land development activities on water quality. I understand and acknowledge that the plan check review of this PDP SWQMP by the City Engineer is confined to a review and does not relieve me, as the Engineer in Responsible Charge of design of storm water BMPs for this project, of my responsibilities for project design.

Patric de Boer RCE 83583 3/31/19
Engineer of Work's Signature, PE Number & Expiration Date

Patric de Boer
Print Name

Omega Engineering Consultants
Company

5/9/17
Date

REGISTERED PROFESSIONAL ENGINEER
PATRIC T. DE BOER
No. C 83583
EXP. 3/31/19
CIVIL
STATE OF CALIFORNIA
Engineer's Stamp

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SUBMITTAL RECORD

Use this Table to keep a record of submittals of this PDP SWQMP. Each time the PDP SWQMP is re-submitted, provide the date and status of the project. In last column indicate changes that have been made or indicate if response to plancheck comments is included. When applicable, insert response to plancheck comments.

Submittal Number	Date	Project Status	Changes
1	8/24/16	☑ Preliminary Design/Planning/CEQA ☐ Final Design	Initial Submittal
2	10/17/16	☑ Preliminary Design/Planning/CEQA ☐ Final Design	Revised 1st plancheck comments
3	Enter a date.	☐ Preliminary Design/Planning/CEQA ☑ Final Design	Click here to enter text.
4	Enter a date.	☐ Preliminary Design/Planning/CEQA ☑ Final Design	Click here to enter text.

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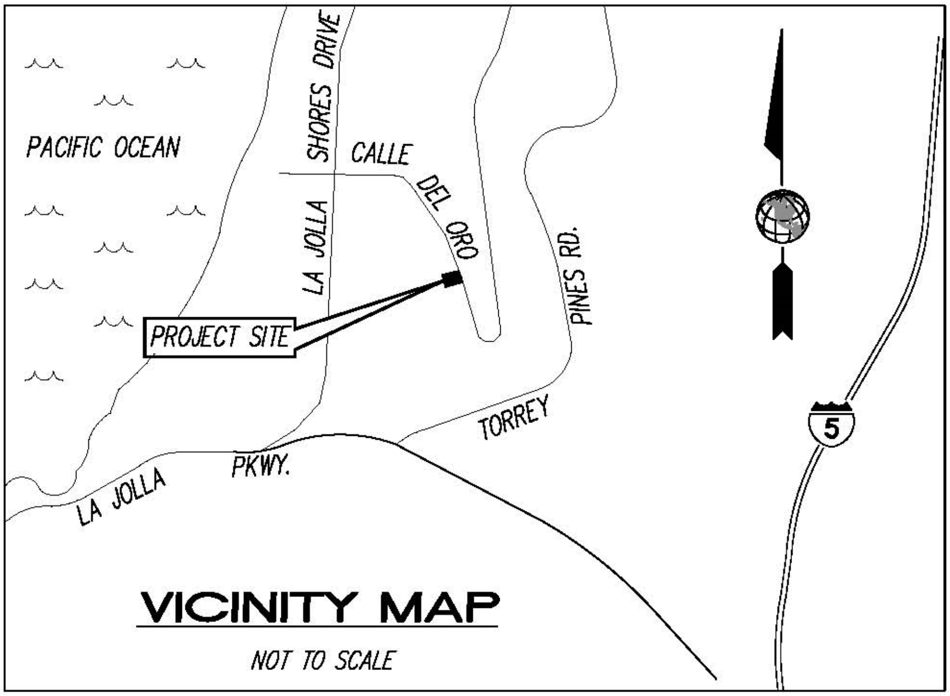
PROJECT VICINITY MAP

Project Name: Klein Residence
Permit Application Number: 441535



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THE CITY OF SAN DIEGO

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

Storm Water Requirements Applicability Checklist

FORM
DS-560
February 2016

Project Address:
2585 Calle Del Oro La Jolla, CA 92037

Project Number *(for the City Use Only)*:
Click here to enter project number

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 administrated 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 that results in ground disturbance and contact with storm water runoff?

   ☒ Yes; WPCP required, skip questions 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 questions 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 one of the following activities and associated curb/ sidewalk repair: water services, sewer lateral, storm drain lateral, or dry 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, curb and gutter replacement, and retaining wall encroachments.

   ☐ Yes; no document required

Check one of the boxes to the right, 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 processes 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 question 1-3, and checked "Yes" for question 4
PART B **does not apply and no document is required. Continue to Section 2.**

More information on the City's construction BMP requirements as well as CGP requirements can be found at: www.sandiego.gov/stormwater/regulations/swguide/constructing.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 Stat e 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. A map of the ASBS watershed can he found here <http://www.swrcb.ca.gov/water_issues/programs/ocean/asbs_map.shtml> ASBS 29 La Jolla

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 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; PDP not exempt. PDP requirements apply.

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 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 that creates 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 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 STORM WATER REQUIREMENTS.** ☐
2. The project is a **STANDARD 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 hydromodification management. ☐

Name of Owner or Agent *(Please Print)*: Patric de Boer
Title: Project Engineer

Signature: Patric de Boer
Date: Insert Date 5/9/17

Applicability of Permanent, Post-Construction Storm Water BMP Requirements

(Storm Water Intake Form for all Development Permit Applications)

Form I-1

Project Identification

Project Name: Klein Residence

Permit Application Number: PTS 441535	Date: 8/22/16

Determination of Requirements

The purpose of this form is to identify permanent, post-construction requirements that apply to the project. This form serves as a short summary of applicable requirements, in some cases referencing separate forms that will serve as the backup for the determination of requirements.

Answer each step below, starting with Step 1 and progressing through each step until reaching "Stop". Refer to Part 1 of Storm Water Standards sections and/or separate forms referenced in each step below.

Step	Answer	Progression
Step 1: Is the project a "development project"? See Section 1.3 of the BMP Design Manual (Part 1 of Storm Water Standards) for guidance.	● Yes	Go to Step 2.
	○ No	Stop. Permanent BMP requirements do not apply. No SWQMP will be required. Provide discussion below.

Discussion / justification if the project is not a "development project" (e.g., the project includes only interior remodels within an existing building):

Project intends to remove the existing house and build a new single family residence in its place.

Step 2: Is the project a Standard Project, Priority Development Project (PDP), or exception to PDP definitions? To answer this item, see Section 1.4 of the BMP Design Manual (Part 1 of Storm Water Standards) in its entirety for guidance, AND complete Storm Water Requirements Applicability Checklist.	○ Standard Project	Stop. Standard Project requirements apply.
	● PDP	PDP requirements apply, including PDP SWQMP. Go to Step 3.
	○ PDP Exempt	Stop. Standard Project requirements apply. Provide discussion and list any additional requirements below.

Discussion / justification, and additional requirements for exceptions to PDP definitions, if applicable:

Click or tap here to enter text.

<table>
<tr><th colspan="3">Form I-1 Page 2</th></tr>
<tr><th>Step</th><th>Answer</th><th>Progression</th></tr>
<tr><td rowspan="2">Step 3. Is the project subject to earlier PDP requirements due to a prior lawful approval?
See Section 1.10 of the BMP Design Manual (Part 1 of Storm Water Standards) for guidance.</td><td>○ Yes</td><td>Consult the City Engineer to determine requirements.
Provide discussion and identify requirements below.
Go to Step 4.</td></tr>
<tr><td>● No</td><td>BMP Design Manual PDP requirements apply.
Go to Step 4.</td></tr>
<tr><td colspan="3">Discussion / justification of prior lawful approval, and identify requirements (<u>not required if prior lawful approval does not apply</u>):
Click or tap here to enter text.</td></tr>
<tr><td rowspan="2">Step 4. Do hydromodification control requirements apply?
See Section 1.6 of the BMP Design Manual (Part 1 of Storm Water Standards) for guidance.</td><td>○ Yes</td><td>PDP structural BMPs required for pollutant control (Chapter 5) and hydromodification control (Chapter 6).
Go to Step 5.</td></tr>
<tr><td>● No</td><td>Stop.
PDP structural BMPs required for pollutant control (Chapter 5) only.
Provide brief discussion of exemption to hydromodification control below.</td></tr>
<tr><td colspan="3">Discussion / justification if hydromodification control requirements do <u>not</u> apply:
Runoff from project drains via hardened conveyance directly to the Pacific Ocean.</td></tr>
<tr><td rowspan="2">Step 5. Does protection of critical coarse sediment yield areas apply?
See Section 6.2 of the BMP Design Manual (Part 1 of Storm Water Standards) for guidance.</td><td>○ Yes</td><td>Management measures required for protection of critical coarse sediment yield areas (Chapter 6.2).
Stop.</td></tr>
<tr><td>● No</td><td>Management measures not required for protection of critical coarse sediment yield areas.
Provide brief discussion below.
Stop.</td></tr>
<tr><td colspan="3">Discussion / justification if protection of critical coarse sediment yield areas does <u>not</u> apply:
Project not located in, or draining to CCSYA.</td></tr>
</table>

Site Information Checklist For PDPs

Form I-3B

Project Summary Information

Project Name	Klein Residence
Project Address	2585 Calle del Oro La Jolla, CA 92037
Assessor's Parcel Number(s) (APN(s))	346-331-03-00
Permit Application Number	PTS 441535
Project Watershed	Select One: ○ San Dieguito River ● Penasquitos ○ Mission Bay ○ San Diego River ○ San Diego Bay ○ Tijuana River
Hydrologic subarea name with Numeric Identifier up to two decimal paces (9XX.XX)	Hydrologic Area: Scripps (906.30) , Subarea: N/A
Project Area (total area of Assessor's Parcel(s) associated with the project or total area of the right-of-way)	0.483 Acres ([SQFT] Square Feet)
Area to be disturbed by the project (Project Footprint)	0.483 Acres (21,073 Square Feet)
Project Proposed Impervious Area (subset of Project Footprint)	0.324 Acres (14,096 Square Feet)
Project Proposed Pervious Area (subset of Project Footprint)	0.160 Acres (6,977 Square Feet)
Note: Proposed Impervious Area + Proposed Pervious Area = Area to be Disturbed by the Project. This may be less than the Project Area.	
The proposed increase or decrease in impervious area in the proposed condition as compared to the pre-project condition.	+33 %

Project Name: Klein Residence

Form I-3B Page 2 of 11
Description of Existing Site Condition and Drainage Patterns
Current Status of the Site (select all that apply): ☒ Existing development ☐ Previously graded but not built out ☐ Agricultural or other non-impervious use ☐ Vacant, undeveloped/natural Description / Additional Information: Existing single family home with pool.
Existing Land Cover Includes (select all that apply): ☒ Vegetative Cover ☒ Non-Vegetated Pervious Areas ☒ Impervious Areas Description / Additional Information: Ex site is 33.8% impervious, wich includes building area, concrete walkways, and driveway area. Pervious cover consists of landscaping, and bare ground.
Underlying Soil belongs to Hydrologic Soil Group (select all that apply): ☐ NRCS Type A ☐ NRCS Type B ☐ NRCS Type C ☒ NRCS Type D
Approximate Depth to Groundwater (GW): ○ GW Depth < 5 feet ○ 5 feet < GW Depth < 10 feet ○ 10 feet < GW Depth < 20 feet ● GW Depth > 20 feet
Existing Natural Hydrologic Features (select all that apply): ☐ Watercourses ☐ Seeps ☐ Springs ☐ Wetlands ☒ None Description / Additional Information: Click or tap here to enter text.

Form I-3B Page 3 of 11

Description of Existing Site Topography and Drainage:

How is storm water runoff conveyed from the site? At a minimum, this description should answer:

1. Whether existing drainage conveyance is natural or urban;
2. If runoff from offsite is conveyed through the site? If yes, quantification of all offsite drainage areas, design flows, and locations where offsite flows enter the project site and summarize how such flows are conveyed through the site;
3. Provide details regarding existing project site drainage conveyance network, including storm drains, concrete channels, swales, detention facilities, storm water treatment facilities, and natural and constructed channels;
4. Identify all discharge locations from the existing project along with a summary of the conveyance system size and capacity for each of the discharge locations. Provide summary of the pre-project drainage areas and design flows to each of the existing runoff discharge locations.

Description / Additional Information:

The existing site is the location of a single family residence, built on a fill slope that slopes down toward the westerly boundary of the site. No natural drainage conveyances exist onsite. Runoff generated by the westerly half of the site flows via sheet flow and concentrated surface flow to the westerly boundary of the site and onto the neighboring site. Runoff generated by the easterly half of the site drains across the driveway of the site and into the gutter of Calle Del Oro.

Runoff from both discharge points is eventually intercepted by the public storm drain system and conveyed to the Pacific Ocean at La Jolla Shores.

The site recieves no run-on from offsite areas. The project hydrology report has found that the existing site discharges 0.60 cfs to the westerly boundary of the site and 0.81 cfs to the easterly boundary at Calle Del Oro for the 100-yr storm. This flow is discharged along the westerly boundary of the site, onto the neighboring lot.

Form I-3B Page 4 of 11

Description of Proposed Site Development and Drainage Patterns

Project Description / Proposed Land Use and/or Activities:

The proposed project will be a single family residence. The onsite flow patterns will be modified to route the majority of the runoff generated by the site to a biofiltration basin and thence to a pump vault. Stormwater will thence be pumped via force main to a discharge point at the the curb along Calle Del Oro The offsite flowpath from this point will be identical to existing conditions. A small portion of the site below a retaining wall along the westerly boundary will still drain to the westerly property line. This area is entirely pervious and is considered a self mitigating area.

The project hydrology report has found that the proposed site discharges 0.14 cfs to the westerly boundary of the site and 0.38 cfs to the easterly boundary at Calle Del Oro for the 100-yr storm. This flow is discharged along the westerly boundary of the site, onto the neighboring lot.

List/describe proposed impervious features of the project (e.g., buildings, roadways, parking lots, courtyards, athletic courts, other impervious features):

The proposed impervious area will consist of PCC or AC driveway, PCC walkways and building roof. The project also proposes to use artificial turf in the backyard. For BMP sizing calculations, this artificatial turf area is assumed to have a C value identical to pavement (0.9)

List/describe proposed pervious features of the project (e.g., landscape areas):

All of the poposed pervious areas of the site will be landscaped. The majority of this area will be located below a retaining wall along the westerly boundary, with the rest being scattered across the site in various planter areas.

Does the project include grading and changes to site topography?

● Yes

○ No

Description / Additional Information:

The redevelopment of the site will result in the disturbance and regrading of the entire site area. The existing discharge points will be maintained.

Form I-3B Page 5 of 11

Does the project include changes to site drainage (e.g., installation of new storm water conveyance systems)?

● Yes

○ No

If yes, provide details regarding the proposed project site drainage conveyance network, including storm drains, concrete channels, swales, detention facilities, storm water treatment facilities, natural and constructed channels, and the method for conveying offsite flows through or around the proposed project site. Identify all discharge locations from the proposed project site along with a summary of the conveyance system size and capacity for each of the discharge locations. Provide a summary of pre and post-project drainage areas and design flows to each of the runoff discharge locations. Reference the drainage study for detailed calculations.

Description / Additional Information:

Site drainage will no longer be facilitated via surface flow to the discharge point. The proposed project will collect storm water in a system of pipes, which will convey it to a biofiltration area in the back yard. Runoff will be treated by filtration through the soil media layer and thence drain through a perforated subdrain to a pump vault. A force main pump will convey stormwater to an outlet point at the easterly boundary of the site along Calle Del Oro. A small portion of the site located below the proposed retaining wall will drain to the westerly boundary. This area is negligible, and as it is entirely pervious landscaping, it shall be considered a self mitigating area in this report.

Project Name: Klein Residence

Form I-3B Page 6 of 11

Identify whether any of the following features, activities, and/or pollutant source areas will be present (select all that apply):

☒ On-site storm drain inlets
☐ Interior floor drains and elevator shaft sump pumps
☒ Interior parking garages
☒ Need for future indoor & structural pest control
☒ Landscape/Outdoor Pesticide Use
☒ Pools, spas, ponds, decorative fountains, and other water features
☐ Food service
☐ Refuse areas
☐ Industrial processes
☐ Outdoor storage of equipment or materials
☐ Vehicle and Equipment Cleaning
☐ Vehicle/Equipment Repair and Maintenance
☐ Fuel Dispensing Areas
☐ Loading Docks
☒ Fire Sprinkler Test Water
☐ Miscellaneous Drain or Wash Water
☐ Plazas, sidewalks, and parking lots
☐ Large Trash Generating Facilities
☐ Animal Facilities
☐ Plant Nurseries and Garden Centers
☐ Automotive-related Uses

Description / Additional Information:
N/A

Form I-3B Page 7 of 11

Identification and Narrative of Receiving Water

Narrative describing flow path from discharge location(s), through urban storm conveyance system, to receiving creeks, rivers, and lagoons and ultimate discharge location to Pacific Ocean (or bay, lagoon, lake or reservoir, as applicable)

Runoff discharged to the gutter along Calle Del Oro flows in a northerly direction down the street until it is intercepted by a public storm drain inlet several hundred yards north of the site. Runoff thence flows through the public MS4 system until it reaches a discharge point to the Pacific Ocean at La Jolla Shores.

Provide a summary of all beneficial uses of receiving waters downstream of the project discharge locations.

Pacific Ocean Beneficial Uses:
-IND, NAV, REC1, REC2, BIOL WILD, RARE, MAR, AQUA, MIGR, SPWN, SHELL

Identify all ASBS (areas of special biological significance) receiving waters downstream of the project discharge locations.

Site drains to the La Jolla Area of Significant Biological Significance (ASBS). The site discharges to an outfall at La Jolla Shores. The discharge is not considered direct because it comingles with runoff from other offsite areas in the public MS4 before it is discharged to the ASBS. the entirety of the site will drain to the bioretention area before flowing offsite. This will prevent non stormwater discahrges from impacting downstream areas.

Provide distance from project outfall location to impaired or sensitive receiving waters.

Site outfalls directly into senstive receiving water (distance = 0 miles)

Sumarize information regarding the proximity of the permanent, post-construction storm water BMPs to the City's Multi-Habitat Planning Area and environmentally sensitive lands

The project site is more than a mile from the nearest MHPA area, which is located near Black's Beach. Runoff from the site does not drain to this area.

Form I-3B Page 8 of 11

Identification of Receiving Water Pollutants of Concern

List any 303(d) impaired water bodies within the path of storm water from the project site to the Pacific Ocean (or bay, lagoon, lake or reservoir, as applicable), identify the pollutant(s)/stressor(s) causing impairment, and identify any TMDLs and/or Highest Priority Pollutants from the WQIP for the impaired water bodies:

303(d) Impaired Water Body	Pollutant(s)/Stressor(s)	TMDLs/ WQIP Highest Priority Pollutant
Pacific Ocean Shoreline	Total Coliform	Est. TMDL completion: 2021
Click or tap here to enter text.	Click or tap here to enter text.	Click or tap here to enter text.
Click or tap here to enter text.	Click or tap here to enter text.	Click or tap here to enter text.
Click or tap here to enter text.	Click or tap here to enter text.	Click or tap here to enter text.
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Click or tap here to enter text.	Click or tap here to enter text.	Click or tap here to enter text.
Click or tap here to enter text.	Click or tap here to enter text.	Click or tap here to enter text.

Identification of Project Site Pollutants*

*Identification of project site pollutants is only required if flow-thru treatment BMPs are implemented onsite in lieu of retention or biofiltration BMPs (note the project must also participate in an alternative compliance program unless prior lawful approval to meet earlier PDP requirements is demonstrated)

Identify pollutants anticipated from the project site based on all proposed use(s) of the site (see BMP Design Manual (Part 1 of Storm Water Standards) Appendix B.6):

Pollutant	Not Applicable to the Project Site	Anticipated from the Project Site	Also a Receiving Water Pollutant of Concern
Sediment	☒	☐	☐
Nutrients	☒	☐	☐
Heavy Metals	☒	☐	☐
Organic Compounds	☒	☐	☐
Trash & Debris	☒	☐	☐
Oxygen Demanding Substances	☒	☐	☐
Oil & Grease	☒	☐	☐
Bacteria & Viruses	☒	☐	☐
Pesticides	☒	☐	☐

Form I-3B Page 9 of 11

Hydromodification Management Requirements

Do hydromodification management requirements apply (see Section 1.6 of the BMP Design Manual)?

○ Yes, hydromodification management flow control structural BMPs required.
○ No, the project will discharge runoff directly to existing underground storm drains discharging directly to water storage reservoirs, lakes, enclosed embayments, or the Pacific Ocean.
● No, the project will discharge runoff directly to conveyance channels whose bed and bank are concrete-lined all the way from the point of discharge to water storage reservoirs, lakes, enclosed embayments, or the Pacific Ocean.
○ No, the project will discharge runoff directly to an area identified as appropriate for an exemption by the WMAA for the watershed in which the project resides.

Description / Additional Information (to be provided if a 'No' answer has been selected above):

Runoff does not encounter any habitat or unstabilized conveyances until the outfall to the Pacific Ocean

Critical Coarse Sediment Yield Areas*

*This Section only required if hydromodification management requirements apply

Based on Section 6.2 and Appendix H does CCSYA exist on the project footprint or in the upstream area draining through the project footprint?

○ Yes
● No, No critical coarse sediment yield areas to be protected based on WMAA maps

Discussion / Additional Information:

No CCSYA are located on, downstream, or upstream of the project site.

Project Name: **Klein Residence**

Form I-3B Page 10 of 11
Flow Control for Post-Project Runoff* *This Section only required if hydromodification management requirements apply
List and describe point(s) of compliance (POCs) for flow control for hydromodification management (see Section 6.3.1). For each POC, provide a POC identification name or number correlating to the project's HMP Exhibit and a receiving channel identification name or number correlating to the project's HMP Exhibit. N/A, No hydromodification controls area requred for this project
Has a geomorphic assessment been performed for the receiving channel(s)? ● No, the low flow threshold is 0.1Q2 (default low flow threshold) ○ Yes, the result is the low flow threshold is 0.1Q2 ○ Yes, the result is the low flow threshold is 0.3Q2 ○ Yes, the result is the low flow threshold is 0.5Q2 If a geomorphic assessment has been performed, provide title, date, and preparer: N/A, No hydromodification controls area requred for this project
Discussion / Additional Information: (optional) N/A, No hydromodification controls area requred for this project

Form I-3B Page 11 of 11

Other Site Requirements and Constraints

When applicable, list other site requirements or constraints that will influence storm water management design, such as zoning requirements including setbacks and open space, or local codes governing minimum street width, sidewalk construction, allowable pavement types, and drainage requirements.

The proposed project is using biofiltration with no infiltration for stormwater treatment. Partial retention is not feasible for this site, as it is located in a steep hillside area, which makes it infeasible to infiltrate without the risk of negative impacts due to soil instability.

Optional Additional Information or Continuation of Previous Sections As Needed

This space provided for additional information or continuation of information from previous sections as needed.

N/A

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Project Name: **Klein Residence**

Source Control BMP Checklist for All Development Projects

Form I-4

Source Control BMPs

All development projects must implement source control BMPs SC-1 through SC-6 where applicable and feasible. See Chapter 4 and Appendix E of the BMP Design Manual (Part 1 of the Storm Water Standards) for information to implement source control BMPs shown in this checklist.

Answer each category below pursuant to the following.

- "Yes" means the project will implement the source control BMP as described in Chapter 4 and/or Appendix E of the BMP Design Manual. Discussion / justification is not required.
- "No" means the BMP is applicable to the project but it is not feasible to implement. Discussion / justification must be provided.
- "N/A" means the BMP is not applicable at the project site because the project does not include the feature that is addressed by the BMP (e.g., the project has no outdoor materials storage areas). Discussion / justification may be provided.

Source Control Requirement	Applied?		
SC-1 Prevention of Illicit Discharges into the MS4	☒ Yes	☐ No	☐ N/A
Discussion / justification if SC-1 not implemented: Click or tap here to enter text.			
SC-2 Storm Drain Stenciling or Signage	☒ Yes	☐ No	☐ N/A
Discussion / justification if SC-2 not implemented: Click or tap here to enter text.			
SC-3 Protect Outdoor Materials Storage Areas from Rainfall, Run-On, Runoff, and Wind Dispersal	☐ Yes	☐ No	☒ N/A
Discussion / justification if SC-3 not implemented: No outdoor storage proposed for this single family residence.			
SC-4 Protect Materials Stored in Outdoor Work Areas from Rainfall, Run-On, Runoff, and Wind Dispersal	☐ Yes	☐ No	☒ N/A
Discussion / justification if SC-4 not implemented: No outdoor work areas proposed for this single family residence			
SC-5 Protect Trash Storage Areas from Rainfall, Run-On, Runoff, and Wind Dispersal	☒ Yes	☐ No	☐ N/A
Discussion / justification if SC-5 not implemented: Click or tap here to enter text.			

Project Name: **Klein Residence**

Form I-4 Page 2 of 2

Source Control Requirement	Applied?
SC-6 Additional BMPs Based on Potential Sources of Runoff Pollutants (must answer for each source listed below)	
On-site storm drain inlets	☑ Yes ☐ No ☐ N/A
Interior floor drains and elevator shaft sump pumps	☐ Yes ☐ No ☑ N/A
Interior parking garages	☑ Yes ☐ No ☐ N/A
Need for future indoor & structural pest control	☑ Yes ☐ No ☐ N/A
Landscape/Outdoor Pesticide Use	☑ Yes ☐ No ☐ N/A
Pools, spas, ponds, decorative fountains, and other water features	☑ Yes ☐ No ☐ N/A
Food service	☐ Yes ☐ No ☑ N/A
Refuse areas	☐ Yes ☑ No ☐ N/A
Industrial processes	☐ Yes ☐ No ☑ N/A
Outdoor storage of equipment or materials	☐ Yes ☐ No ☑ N/A
Vehicle/Equipment Repair and Maintenance	☐ Yes ☐ No ☑ N/A
Fuel Dispensing Areas	☐ Yes ☐ No ☑ N/A
Loading Docks	☐ Yes ☐ No ☑ N/A
Fire Sprinkler Test Water	☑ Yes ☐ No ☐ N/A
Miscellaneous Drain or Wash Water	☐ Yes ☐ No ☑ N/A
Plazas, sidewalks, and parking lots	☐ Yes ☐ No ☑ N/A
SC-6A: Large Trash Generating Facilities	☐ Yes ☐ No ☑ N/A
SC-6B: Animal Facilities	☐ Yes ☐ No ☑ N/A
SC-6C: Plant Nurseries and Garden Centers	☐ Yes ☐ No ☑ N/A
SC-6D: Automotive-related Uses	☐ Yes ☐ No ☑ N/A

Discussion / justification if SC-6 not implemented. Clearly identify which sources of runoff pollutants are discussed. Justification must be provided for all "No" answers shown above.

A. All on-site storm drain inlets will be marked with the words “No Dumping” or similar

C. Interior parking garage drains will be plumbed to the sewer

D1. Integrated pest management information will be provided to the owners

D2. Landscape will be drought tolerant and plants will be chosen to require a minimal amount of pesticides

E. A sewer connection will be located within hose distance to drain any proposed pools.

G. Protection of single family residence trashcans from wind dispersal and run-on will be the responsibilty of the owners. Owners will provide an adequate number of receptacles of their own discretion. This BMP will not be part of this SWQMP.

N. Fire sprinkler systems will be drained to sanitary sewer

All remaining BMPs are marked N/A because it is not applicable to the project.

Project Name: **Klein Residence**

Site Design BMP Checklist for All Development Projects	Form I-5

Site Design BMPs

All development projects must implement site design BMPs SD-1 through SD-8 where applicable and feasible. See Chapter 4 and Appendix E of the BMP Design Manual (Part 1 of Storm Water Standards) for information to implement site design BMPs shown in this checklist.

Answer each category below pursuant to the following.

- "Yes" means the project will implement the site design BMP as described in Chapter 4 and/or Appendix E of the BMP Design Manual. Discussion / justification is not required.
- "No" means the BMP is applicable to the project but it is not feasible to implement. Discussion / justification must be provided.
- "N/A" means the BMP is not applicable at the project site because the project does not include the feature that is addressed by the BMP (e.g., the project site has no existing natural areas to conserve). Discussion / justification may be provided.

A site map with implemented site design BMPs must be included at the end of this checklist.

Site Design Requirement	Applied?		
SD-1 Maintain Natural Draiange Pathways and Hydrologic Features	☐ Yes	☐ No	☑ N/A
Discussion / justification if SD-1 not implemented: No natural drainage pathways exist on this previously developed site.			
1-1 Are existing natural drainage pathways and hydrologic features mapped on the site map?	☐ Yes	☐ No	☑ N/A
1-2 Are street trees implemented? If yes, are they shown on the site map?	☐ Yes	☑ No	☐ N/A
1-3 Implemented street trees meet the design criteria in SD-1 Fact Sheet (e.g. soil volume, maximum credit, etc.)?	☐ Yes	☑ No	☐ N/A
1-4 Is street tree credit volume calculated using Appendix B.2.2.1 and SD-1 Fact Sheet in Appendix E?	☐ Yes	☑ No	☐ N/A
SD-2 Have natural areas, soils and vegetation been conserved?	☐ Yes	☑ No	☐ N/A
Discussion / justification if SD-2 not implemented: No natural area, or vegetation on this previously developed site.			

Form I-5 Page 2 of 4

Site Design Requirement	Applied?		
SD-3 Minimize Impervious Area	☒ Yes	☐ No	☐ N/A
Discussion / justification if SD-3 not implemented: Click or tap here to enter text.			
SD-4 Minimize Soil Compaction	☒ Yes	☐ No	☐ N/A
Discussion / justification if SD-4 not implemented: Click or tap here to enter text.			
SD-5 Impervious Area Dispersion	☐ Yes	☒ No	☐ N/A
Discussion / justification if SD-5 not implemented: Proposed impervious areas (house, walkways, driveways etc) will cover the majority of the site. These areas will drain to a pervious bioretention area, but not to an area with the design criteria from SD-5. No dispersion credit is being claimed.			
5-1 Is the pervious area receiving runon from impervious area identified on the site map?	☒ Yes	☐ No	
5-2 Does the pervious area satisfy the design criteria in SD-5 Fact Sheet in Appendix E (e.g. maximum slope, minimum length, etc.)	☐ Yes	☒ No	
5-3 Is impervious area dispersion credit volume calculated using Appendix B.2.1.1 and SD-5 Fact Sheet in Appendix E?	☐ Yes	☒ No	

Project Name: **Klein Residence**

Form I-5 Page 3 of 4

Site Design Requirement	Applied?		
SD-6 Runoff Collection	☑ Yes	☐ No	☐ N/A
Discussion / justification if SD-6 not implemented: Click or tap here to enter text.			
6a-1 Are green roofs implemented in accordance with design criteria in SD-6A Fact Sheet? If yes, are they shown on the site map?	☐ Yes	☑ No	☐ N/A
6a-2 Is green roof credit volume calculated using Appendix B.2.1.2 and SD-6A Fact Sheet in Appendix E?	☐ Yes	☑ No	☐ N/A
6b-1 Are permeable pavements implemented in accordance with design criteria in SD-6B Fact Sheet? If yes, are they shown on the site map?	☐ Yes	☑ No	☐ N/A
6b-2 Is permeable pavement credit volume calculated using Appendix B.2.1.3 and SD-6B Fact Sheet in Appendix E?	☐ Yes	☑ No	☐ N/A
SD-7 Landscaping with Native or Drought Tolerant Species	☑ Yes	☐ No	☐ N/A
Discussion / justification if SD-7 not implemented: Click or tap here to enter text.			
SD-8 Harvesting and Using Precipitation	☐ Yes	☑ No	☐ N/A
Discussion / justification if SD-8 not implemented: Site demand is less than 25% of the DVC. Full and partial harvest/reuse is infeasible.			
8-1 Are rain barrels implemented in accordance with design criteria in SD-8 Fact Sheet? If yes, are they shown on the site map?	☐ Yes	☑ No	☐ N/A
8-2 Is rain barrel credit volume calculated using Appendix B.2.2.2 and SD-8 Fact Sheet in Appendix E?	☐ Yes	☑ No	☐ N/A

Project Name: **Klein Residence**

Form I-5 Page 4 of 4
Insert Site Map with all site design BMPs identified:
See Map Pocket, Next Page

Summary of PDP Structural BMPs	Form I-6
PDP Structural BMPs	
All PDPs must implement structural BMPs for storm water pollutant control (see Chapter 5 of the BMP Design Manual, Part 1 of Storm Water Standards). Selection of PDP structural BMPs for storm water pollutant control must be based on the selection process described in Chapter 5. PDPs subject to hydromodification management requirements must also implement structural BMPs for flow control for hydromodification management (see Chapter 6 of the BMP Design Manual). Both storm water pollutant control and flow control for hydromodification management can be achieved within the same structural BMP(s). PDP structural BMPs must be verified by the City at the completion of construction. This includes requiring the project owner or project owner's representative to certify construction of the structural BMPs (complete Form DS-563). PDP structural BMPs must be maintained into perpetuity (see Chapter 7 of the BMP Design Manual). Use this form to provide narrative description of the general strategy for structural BMP implementation at the project site in the box below. Then complete the PDP structural BMP summary information sheet (page 3 of this form) for each structural BMP within the project (copy the BMP summary information page as many times as needed to provide summary information for each individual structural BMP).	
Describe the general strategy for structural BMP implementation at the site. This information must describe how the steps for selecting and designing storm water pollutant control BMPs presented in Section 5.1 of the BMP Design Manual were followed, and the results (type of BMPs selected). For projects requiring hydromodification flow control BMPs, indicate whether pollutant control and flow control BMPs are integrated or separate. The soil on this project site was deemed infeasible for full or partial infiltration conditions due to geologic concerns related to the steep topography of the site. The project is located in a steep hillside overlay and has slopes of 25%. Infiltration would result in slope instability, potential seepage surface seepage and other negative geotechnical impacts. Because of this, fully lined biofiltration was chosen for stormwater treatment. No flow control or hydromodification controls are required for this site as it drains via hardened conveyance directly to the outfall to the Pacific Ocean. (Continue on page 2 as necessary.)	

Form I-6 Page 2 of X
(Page reserved for continuation of description of general strategy for structural BMP implementation at the site)
(Continued from page 1) N/A

Form I-6 Page 3 of X (Copy as many as needed)

Structural BMP Summary Information

Structural BMP ID No. BMP-1

Construction Plan Sheet No. Sheet Number #1

Type of structural BMP:

○ Retention by harvest and use (HU-1)

○ Retention by infiltration basin (INF-1)

○ Retention by bioretention (INF-2)

○ Retention by permeable pavement (INF-3)

○ Partial retention by biofiltration with partial retention (PR-1)

● Biofiltration (BF-1)

○ Flow-thru treatment control with prior lawful approval to meet earlier PDP requirements (provide (BMP type/description in discussion section below)

○ Flow-thru treatment control included as pre-treatment/forebay for an onsite retention or biofiltration BMP (provide BMP type/description and indicate which onsite retention or biofiltration BMP it serves in discussion section below)

○ Flow-thru treatment control with alternative compliance (provide BMP type/description in discussion

○ Detention pond or vault for hydromodification management

○ Other (describe in discussion section below)

Purpose:

● Pollutant control only

○ Hydromodification control only

○ Combined pollutant control and hydromodification control

○ Pre-treatment/forebay for another structural BMP

○ Other (describe in discussion section below)

Who will certify construction of this BMP? Provide name and contact information for the party responsible to sign BMP verification form DS-563	Andrew J. Kann
Who will be the final owner of this BMP?	The owner of the BMP will be the owner of the property, which currently is The Trevor and Staci Klein Trust
Who will maintain this BMP into perpetuity?	Maintenance of the BMP will be done by the owner of the property, which is currently The Trevor and Staci Klein Trust.
What is the funding mechanism for maintenance?	Funding for maintentance will be provided by the property owner

Form I-6 Page 4 of X (Copy as many as needed)

Structural BMP ID No. N/A

Construction Plan Sheet No. N/A

Discussion (as needed):
N/A

THE CITY OF SAN DIEGO

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

Permenant BMP Construction

Self Certification Form

FORM
DS-563
January 2016

Date Prepared: Click here to enter text.

Project No.: Click here to enter text.

Project Applicant: Click here to enter text.

Phone: Click here to enter text.

Project Address: Click here to enter text.

Project Engineer: Click here to enter text.

Phone: Click here to enter text.

The purpose of this form is to verify that the site improvements for the project, identified above, have been constructed in conformance with the approved Storm Water Quality Management Plan (SWQMP) documents and drawings.

This form must be completed by the engineer and submitted prior to final inspection of the construction permit. Completion and submittal of this form is required for all new development and redevelopment projects in order to comply with the City's Storm Water ordinances and NDPES Permit Order No. R9-2013-0001 as amended by R9-2015-0001 and R9-2015-0100. Final inspection for occupancy and/or release of grading or public improvement bonds may be delayed if this form is not submitted and approved by the City of San Diego.

CERTIFICATION:

As the professional in responsible charge for the design of the above project, I certify that I have inspected all constructed Low Impact Development (LID) site design, source control and structural BMP's required per the approved SWQMP and Construction Permit No. Click here to enter text.; and that said BMP's have been constructed in compliance with the approved plans and all applicable specifications, permits, ordinances and Order No. R9-2013-0001 as amended by R9-2015-0001 and R9-2015-0100 of the San Diego Regional Water Quality Control Board.

I understand that this BMP certification statement does not constitute an operation and maintenance verification.

Signature: ________________________________

Date of Signature: _ Insert Date___

Printed Name: _Click here to enter text._

Title: _Click here to enter text._

Phone No. _Click here to enter text._

Engineer's Stamp

DS-563 (12-15)

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ATTACHMENT 1
BACKUP FOR PDP POLLUTANT CONTROL BMPS

This is the cover sheet for Attachment 1.

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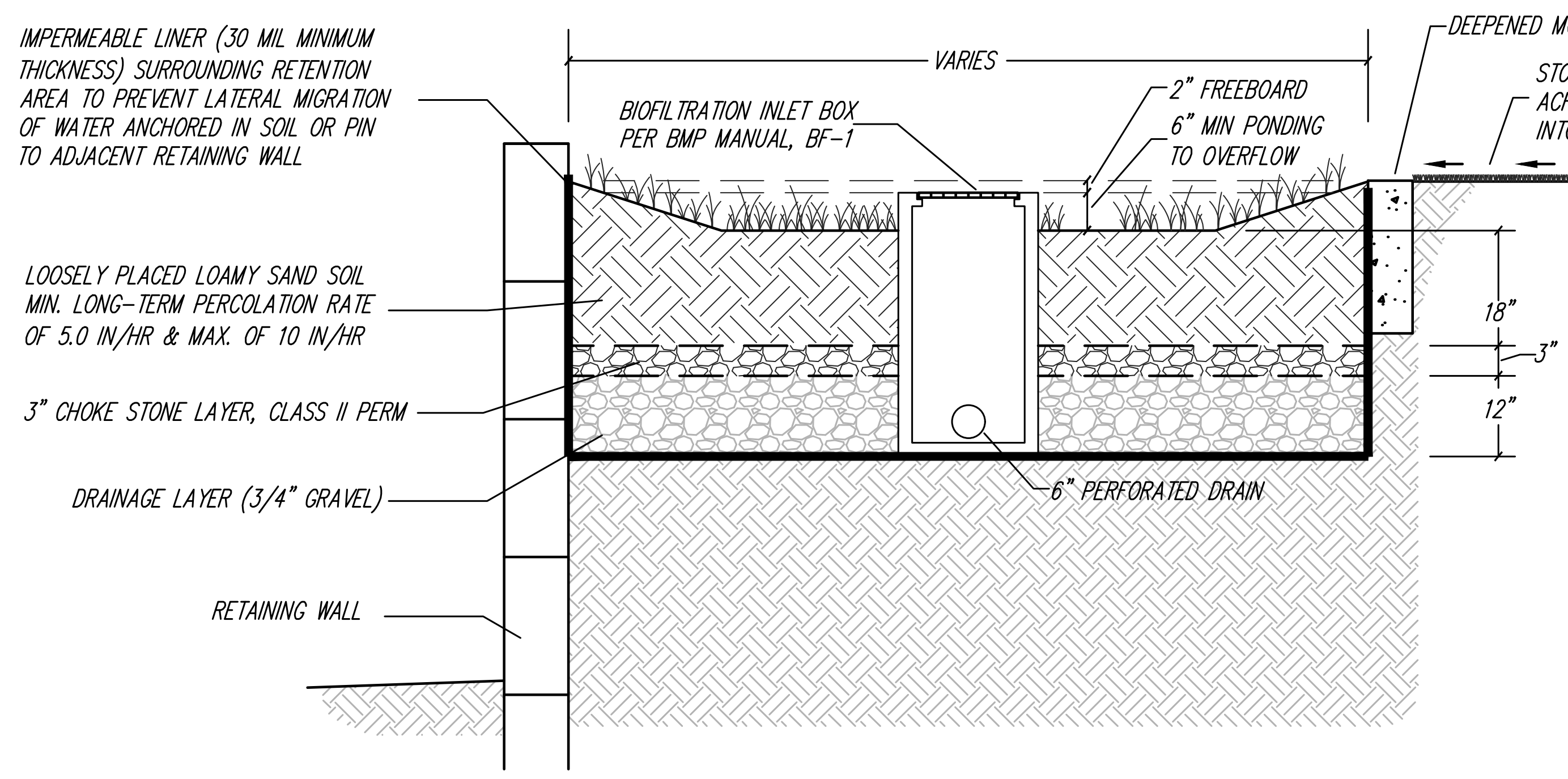


BIOFILTRATION CROSS SECTION

NOT TO SCALE

DMA DATA TABLE

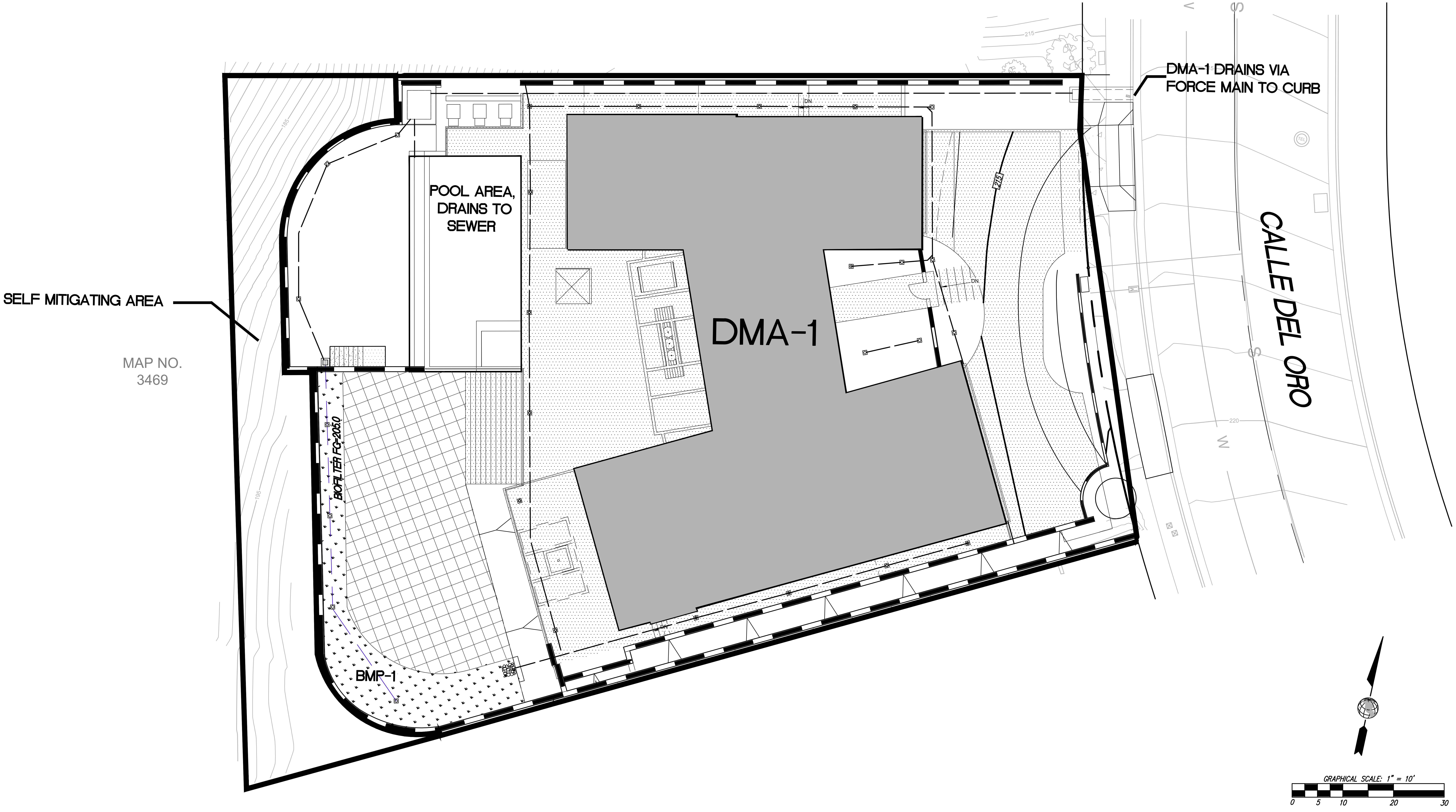
DMA-NO.	TOT. AREA	IMPERVIOUS AREA	IMPERVIOUS %	TYPE/TREATED BY
DMA-1	16,784 SF	14,101 SF	84.0%	IMP-1, BIOFILTRATION AREA
–	–	–	–	–

STRUCTURAL BMP DATA TABLE

IMP-#	TREATING	IMP TYPE	REQ'D FOOTPRINT	PROPOSED FOOTPRINT
IMP-1	DMA-1	BIOFILTRATION	402 SF	600 SF
–	–	–	–	–

LEGEND:

BASIN BOUNDARY	━━
DRAINAGE ARROWS	→ →
DRAINAGE MANAGEMENT AREA NO.	DMA-#
SELF-MITIGATING AREA NO.	SMA-#
INTEGRATED MANAGEMENT PRACTICE (IMP)	IMP-#
BUILDING AREA	▩
HARDSCAPE AREA (IMPERVIOUS)	▒
LANDSCAPED AREA	▭
ARTIFICIAL TURF	☒
BIOFILTRATION/PONDING AREA	▨



Alcorn & Benton
ARCHITECTS
7757 Girard Avenue, La Jolla, California 92037
P.858.459.0805 F.858.459.1350

OMEGA
ENGINEERING CONSULTANTS
4340 VIEWRIDGE AVE, SUITE B
SAN DIEGO, CALIFORNIA 92123
Phone: (858) 634-8620

KLEIN
RESIDENCE

2585 Calle Del Oro
La Jolla, California 92037

Date	December 8, 2016
Project No.	0357
Design/ Drawing	SS/NS
Sheet	C-3
Scale	1"=10'

DETAILS

1.4

ANDREW J. KANN
No. 50940
Exp. 9-30-17
CIVIL

Project Name: **Klein Residence**

Indicate which Items are Included:

Attachment Sequence	Contents	Checklist
Attachment 1a	DMA Exhibit (Required) See DMA Exhibit Checklist.	☒ Included
Attachment 1b	Tabular Summary of DMAs Showing DMA ID matching DMA Exhibit, DMA Area, and DMA Type (Required)* *Provide table in this Attachment OR on DMA Exhibit in Attachment 1a	● Included on DMA Exhibit in Attachment 1a ○ Included as Attachment 1b, separate from DMA Exhibit
Attachment 1c	Form I-7, Harvest and Use Feasibility Screening Checklist (Required unless the entire project will use infiltration BMPs) Refer to Appendix B.3-1 of the BMP Design Manual to complete Form I-7.	● Included ○ Not included because the entire project will use infiltration BMPs
Attachment 1d	Form I-8, Categorization of Infiltration Feasibility Condition (Required unless the project will use harvest and use BMPs) Refer to Appendices C and D of the BMP Design Manual to complete Form I-8.	● Included ○ Not included because the entire project will use harvest and use BMPs
Attachment 1e	Pollutant Control BMP Design Worksheets / Calculations (Required) Refer to Appendices B and E of the BMP Design Manual for structural pollutant control BMP design guidelines and site design credit calculations	☒ Included

Harvest and Use Feasibility Checklist

Form I-7

1. Is there a demand for harvested water (check all that apply) at the project site that is reliably present during the wet season?

☒ Toilet and urinal flushing
☒ Landscape irrigation
☐ Other:_______________

2. If there is a demand; estimate the anticipated average wet season demand over a period of 36 hours. Guidance for planning level demand calculations for toilet/urinal flushing and landscape irrigation is provided in Section B.3.2.

The estimated occupancy of the single family residence is 4 people. Per Table B.3-1 of the BMP design manual, the toilet/urinal use per capita per day is 18.5 gallons. This means the estimated toilet./urinal use is ***4 x 18.5 x 1.5 days = 111 gallons*** in the 36 hours following a qualifying rain event.

The proposed landscaping will be drought tolerant and will not require any irrigation in the 36 hours following a qualifying rain event.

The estimated demand for harvest and reuse is 111 gallons or 14.8 cubic feet.
This is less than 25% of the estimated DCV of **579 cubic feet**

3. Calculate the DCV using worksheet B-2.1.

DCV=**579 cubic feet**

3a. Is the 36 hour demand greater than or equal to the DCV? Yes / No ⇨ ⇩	3b. Is the 36 hour demand greater than 0.25DCV but less than the full DCV? Yes / No ⇨ ⇩	3c. Is the 36 hour demand less than 0.25DCV? Yes ⇩
Harvest and use appears to be feasible. Conduct more detailed evaluation and sizing calculations to confirm that DCV can be used at an adequate rate to meet drawdown criteria.	Harvest and use may be feasible. Conduct more detailed evaluation and sizing calculations to determine feasibility. Harvest and use may only be able to be used for a portion of the site, or (optionally) the storage may need to be upsized to meet long term capture targets while draining in longer than 36 hours.	Harvest and use is considered to be infeasible.

Is harvest and use feasible based on further evaluation?

☐Yes, refer to Appendix E to select and size harvest and use BMP.

☑No, select alternate BMPs.

Klein 13-10407

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition	Worksheet C.4-1

Part 1 - Full Infiltration Feasibility Screening Criteria
Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
1	Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	

Provide basis:

Measured infiltration rate on the site was 0.818 inches per hour with a minimum factor of safety of 2 applied. Although the measured infiltration rate is considered to be appreciable, the geologic conditions on the site are considered unfavorable due to the silty and clayey nature of the formational soils increasing the potential for slope instability and other geotechnical related hazards. In addition, the highly fractured nature of the Ardath Shale underlying the proposed infiltration basin allowed for higher than normal infiltration rates typical of this silty/clayey formation. Simple open pit testing was performed at 1 locations on the site within the proposed infiltration basins in accordance with Appendix D of the City of San Diego BMP design manual. In addition, a comprehensive evaluation of the site was conducted in accordance with Appendix C.2. Please refer to our "Update Report of Limited Geotechnical Investigation" dated September 21, 2016 and our "Update Letter Response to City Comments" dated December 1, 2016 for details of the comprehensive evaluation and investigation conducted, simple open pit test results and converted simple open pit test results to infiltration rate calculations, and maps representative of the study.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

2	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X

Provide basis:

The infiltration test results below the proposed facility locations was 0.818 inches per hour with a minimum factor of safety of 2 applied. In our opinion, any long term infiltration at the site will result in geotechnical hazards which cannot be reasonably mitigated to an acceptable level. Based on our comprehensive evaluation in accordance with Appendix C.2, the anticipated geotechnical hazards are outlined below:

C.2.3 Slope Stability - The formational soils underlying the site consist of sandy silts and sandy clays of the Ardath Shale Formation. The City of San Diego Geologic Hazards Map Sheet 29 indicates the site is located in geologic hazard category 26 "Ardath: unfavorable geologic structure". Any attempt at infiltration on the site may increase the risk of slope failure on the westerly and northerly descending slope, potentially damaging property and life on the site and adjacent properties to the north.

C.2.4 Utility Considerations - Water intrusion into existing utility lines and vaults both on-site and off-site including the adjacent public street (Calle Del Oro) and existing residence which bounds the western and northern property boundary at a lower elevations.

C.2.6 Retaining Walls and Foundations - Water migration under proposed building foundations, proposed retaining walls and existing north neighbor retaining wall resulting in increased lateral pressures and reduction in soil strength.

Please refer to our "Update Report of Limited Geotechnical Investigation" dated September 21, 2016 and our "Update Letter Response to City Comments" dated December 1, 2016 for details of the comprehensive evaluation and investigation conducted, simple open pit test results and converted simple open pit test results to infiltration rate calculations, and maps representative of the study.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

Klein 13-10407

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 2 of 4</th></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td></td><td>☒</td></tr>
<tr><td colspan="4">Provide basis:
In review of our "Report of Limited Geotechnical Investigation" dated February 10, 2014, groundwater was not encountered to a depth of 11.5 feet below existing ground surface. Although groundwater was not encountered to the aforementioned depth, the risk for groundwater related concerns include shallow perched seepage due to the practically impermeable, hard, nature of the silty and clayey formational soils across the site. Based on our comprehensive evaluation in accordance with Appendix C.3, the "other factors" associated with the risk of groundwater contamination include shallow perched seepage that cannot be mitigated to an acceptable level. The anticipated risk for groundwater related concerns are outlined below:
C.3.7 Other Factors - The highly fractured nature of the formational soils in the vicinity of the proposed infiltration basin and the geologic structure observed in our test pit indicate bedding of N60W, 3 degrees NE in the direction of the neighboring property to the north would ultimately create water related hazards and associated problems for the adjacent property. Please refer to our "Update Report of Limited Geotechnical Investigation" dated September 21, 2016 and our "Update Letter Response to City Comments" dated December 1, 2016for details of the comprehensive evaluation and investigation conducted, simple open pit test results and converted simple open pit test results to infiltration rate calculations, and maps representative of the study.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>☒</td><td></td></tr>
<tr><td colspan="4">Provide basis:
No ephemeral streams exist between the project and ocean. The infiltration of storm water would not be expected to cause downstream water balance issues.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result*</td><td colspan="2">If all answers to rows 1 - 4 are "Yes" a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration
If any answer from row 1-4 is "No", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2</td><td>No Infiltration</td></tr>
</table>

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

Klein 13-10407

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 3 of 4</th></tr>
<tr><td colspan="4">Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria
Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?</td></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>5</td><td>Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:
Measured infiltration rate on the site was 0.818 inches per hour with a minimum factor of safety of 2 applied. Although the measured infiltration rate is considered to be appreciable, the geologic conditions on the site are considered unfavorable due to the silty and clayey nature of the formational soils increasing the potential for slope instability and other geotechnical related hazards. In addition, the fractured nature of the Ardath Shale underlying the proposed infiltration basin allowed for higher than normal infiltration rates typical of this silty/clayey formation. Please refer to our "Update Report of Limited Geotechnical Investigation" dated September 21, 2016 and our "Update Letter Response to City Comments" dated December 1, 2016 for details of the comprehensive evaluation and investigation conducted, simple open pit test results and converted simple open pit test results to infiltration rate calculations, and maps representative of the study.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>6</td><td>Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis:
In our opinion, any long term infiltration at the site will result in geotechnical hazards which cannot be reasonably mitigated to an acceptable level. Based on our comprehensive evaluation in accordance with Appendix C.2, the anticipated geotechnical hazards are outlined below:
C.2.3 Slope Stability - The formational soils underlying the site consist of sandy silts and sandy clays of the Ardath Shale Formation. The City of San Diego Geologic Hazards Map Sheet 29 indicates the site is located in geologic hazard category 26 "Ardath: unfavorable geologic structure". Any attempt at infiltration on the site may increase the risk of slope failure on the westerly and northerly descending slopes, potentially damaging property and life on the site and adjacent properties.
C.2.4 Utility Considerations - Water intrusion into existing utility lines and vaults both on-site and off-site including the adjacent public street (Calle Del Oro) and existing development which bounds the western and northern property boundary at a lower elevation.
C.2.6 Retaining Walls and Foundations - Water migration under proposed building foundations and proposed retaining walls and existing north neighbor retaining wall resulting in increased lateral pressures and reduction in soil strength.
Please refer to our "Update Report of Limited Geotechnical Investigation" dated September 21, 2016 and our "Update Letter Response to City Comments" dated December 1, 2016 for details of the comprehensive evaluation and investigation conducted, simple open pit test results and converted simple open pit test results to infiltration rate calculations, and maps representative of the study.
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
</table>

Klein 13-10407

Worksheet C.4-1 Page 4 of 4

Criteria	Screening Question	Yes	No
7	Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X

Provide basis:

In review of our "Report of Limited Geotechnical Investigation" dated February 10, 2014, groundwater was not encountered to a depth of 11.5 feet below existing ground surface. Although groundwater was not encountered to the aforementioned depth, the risk for groundwater related concerns include shallow perched seepage due to the practically impermeable hard nature of the silty and clayey formational soils across the site. Based on our comprehensive evaluation in accordance with Appendix C.3, the "other factors" associated with the risk of groundwater contamination include shallow perched seepage that cannot be mitigated to an acceptable level. The anticipated risk for groundwater related concerns are outlined below:

C.3.7 Other Factors

The highly fractured nature of the formational soils in the vicinity of the proposed infiltration basin and the geologic structure observed in our test pit indicate bedding of N60W, 3 degrees NE in the direction of the neighboring property to the north would ultimately create water related hazards and associated problems for the adjacent property.

Please refer to our "Update Report of Limited Geotechnical Investigation" dated September 21, 2016 and our "Update Letter Response to City Comments" dated December 1, 2016 for details of the comprehensive evaluation and investigation conducted, simple open pit test results and converted simple open pit test results to infiltration rate calculations, and maps representative of the study.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Criteria	Screening Question	Yes	No
8	Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	

Provide basis:

Infiltration of storm water would not violate downstream water rights. Project drains directly via hardened MS-4 to the Pacific Ocean. No water ways or water bodies exist between the project and the ultimate outflow to the ocean.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Criteria	Screening Question	Yes	No
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration. If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.		No Infiltration

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings

Worksheet B.2-1 DCV

Design Capture Volume		Worksheet B.2-1		
1	85th percentile 24-hr storm depth from Figure B.1-1	d=	.51	inches
2	Area tributary to BMP (s)	A=	.385	acres
3	Area weighted runoff factor (estimate using Appendix B.1.1 and B.2.1)	C=	0.81	unitless
4	Trees Credit Volume	TCV=	0	cubic-feet
5	Rain barrels Credit Volume	RCV=	0	cubic-feet
6	Calculate DCV = (3630 x C x d x A) – TCV - RCV	DCV=	574	cubic-feet

- See Calculation table for details

DCV CALCULATIONS

DMA DATA TABLE

DMA No.	Area (sf)	Impervious %	Impervious C factor	Pervious C factor	C	Weighted Impervious Area	Weighted Pervious Area (sf)	Total Weighted Area (sf)	85th % storm depth (in)	Capture Volume (cf)
DMA-1	16,784	84.0	0.9	0.30	0.80	12,689	806	13,494	0.51	574
-										
-										
-										
-										
-										
-										
TOTAL	16,784	84.0	0.9	0.30	0.80	12,689	806	13,494	0.51	574

$$Total\ Weighted\ Area = (0.30 \times pervious\ area) + (0.90\ \times impervious\ area)$$

$$85th\ percentile\ Storm\ Depth\ from\ County\ Isopluvial\ Map$$

$$Capture\ Volume = Storm\ Depth\ \times Total\ Weighted\ Area$$

DMA-1 TREATED BY BIO-1

Worksheet B.5-1: Simple Sizing Method for Biofiltration BMPs

Simple Sizing Method for Biofiltration BMPs		Worksheet B.5-1 (Page 1 of 2)	
1	Remaining DCV after implementing retention BMPs	574	cubic-feet
Partial Retention			
2	Infiltration rate from Worksheet D.5-1 if partial infiltration is feasible	0	in/hr.
3	Allowable drawdown time for aggregate storage below the underdrain	36	hours
4	Depth of runoff that can be infiltrated [Line 2 x Line 3]	0	inches
5	Aggregate pore space	0.40	in/in
6	Required depth of gravel below the underdrain [Line 4/ Line 5]	0	inches
7	Assumed surface area of the biofiltration BMP	500	sq-ft
8	Media retained pore storage	0.1	in/in
9	Volume retained by BMP [[Line 4 + (Line 12 x Line 8)]/12] x Line 7	75	cubic-feet
10	DCV that requires biofiltration [Line 1 – Line 9]	499	cubic-feet
BMP Parameters			
11	Surface Ponding [6 inch minimum, 12 inch maximum]	6	inches
12	Media Thickness [18 inches minimum], also add mulch layer thickness to this line for sizing calculations	18	inches
13	Aggregate Storage above underdrain invert (12 inches typical) – use 0 inches for sizing if the aggregate is not over the entire bottom surface area	12	inches
14	Freely drained pore storage	0.2	in/in
15	Media filtration rate to be used for sizing (5 in/hr. with no outlet control; if the filtration rate is controlled by the outlet use the outlet controlled rate which will be less than 5 in/hr.)	5	in/hr.
Baseline Calculations			
16	Allowable Routing Time for sizing	6	hours
17	Depth filtered during storm [Line 15 x Line 16]	30	inches
18	Depth of Detention Storage [Line 11 + (Line 12 x Line 14) + (Line 13 x Line 5)]	14.4	inches
19	Total Depth Treated [Line 17 + Line 18]	44.4	inches

Note: Line 7 is used to estimate the amount of volume retained by the BMP. Update assumed surface area in Line 7 until its equivalent to the required biofiltration footprint (either Line 21 or Line 23)

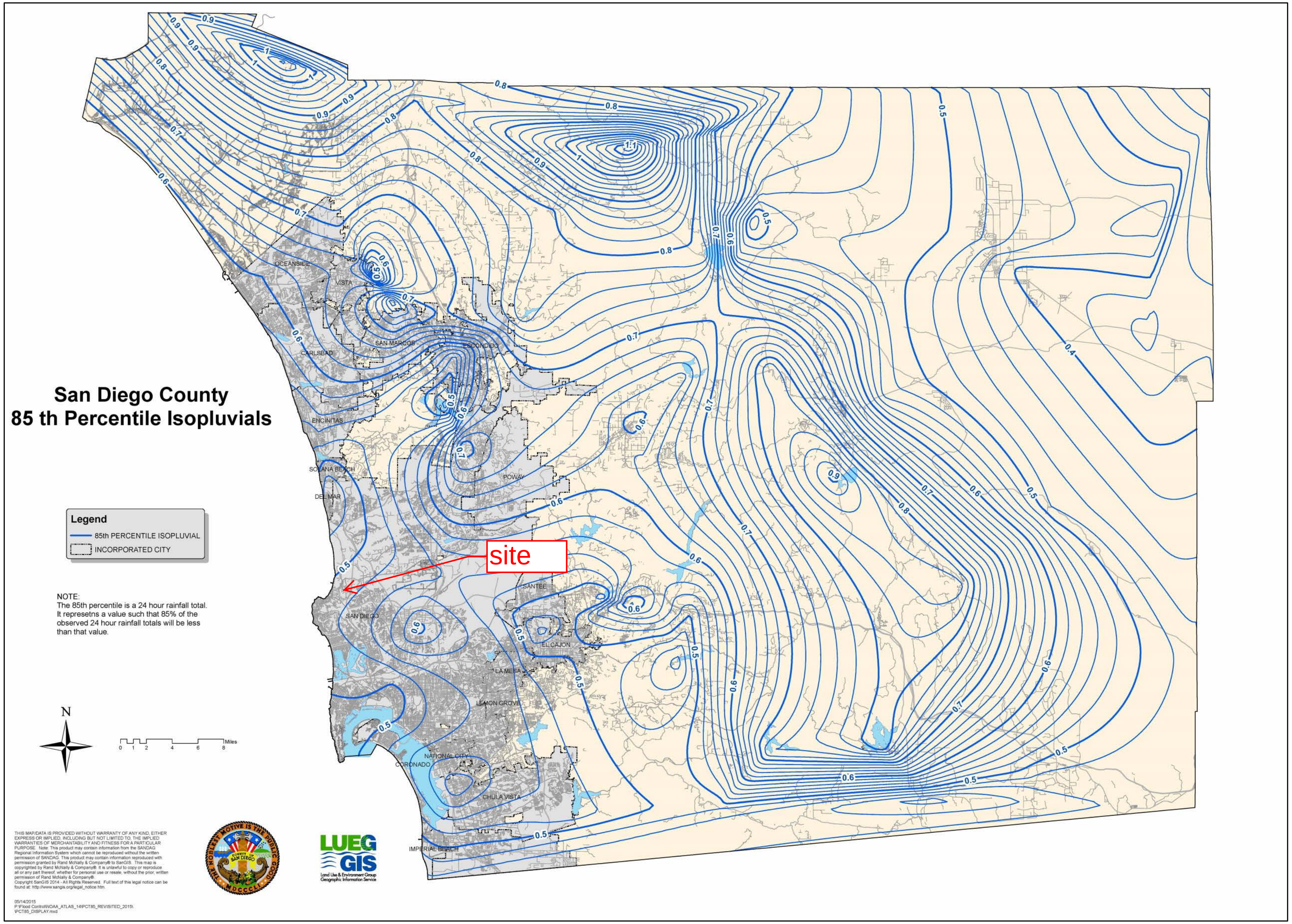
Worksheet B.5-1: Simple Sizing Method for Biofiltration BMPs (continued)

	Simple Sizing Method for Biofiltration BMPs	Worksheet B.5-1 (Page 2 of 2)	
Option 1 – Biofilter 1.5 times the DCV			
20	Required biofiltered volume [1.5 x Line 10]	749	cubic-feet
21	Required Footprint [Line 20/ Line 19] x 12	204	sq-ft
Option 2 - Store 0.75 of remaining DCV in pores and ponding			
22	Required Storage (surface + pores) Volume [0.75 x Line 10]	374	cubic-feet
23	Required Footprint [Line 22/ Line 18] x 12	311	sq-ft
Footprint of the BMP			
24	Area draining to the BMP	16,784	sq-ft
25	Adjusted Runoff Factor for drainage area (Refer to Appendix B.1 and B.2)	0.80	
26	BMP Footprint Sizing Factor (Default 0.03 or an alternative minimum footprint sizing factor from Worksheet B.5-2, Line 11)	.03	
27	Minimum BMP Footprint [Line 24 x Line 25 x Line 26]	402	sq-ft
28	Footprint of the BMP = Maximum(Minimum(Line 21, Line 23), Line 27)	402	sq-ft
Check for Volume Reduction [Not applicable for No Infiltration Condition]			
29	Calculate the fraction of DCV retained in the BMP [Line 9/Line 1]	N/A	unitless
30	Minimum required fraction of DCV retained for partial infiltration condition	0.375	unitless
31	Is the retained DCV ≥ 0.375? If the answer is no increase the footprint sizing factor in Line 26 until the answer is yes for this criterion.	☐ Yes	☐ No

Note:

1. Line 7 is used to estimate the amount of volume retained by the BMP. Update assumed surface area in Line 7 until its equivalent to the required biofiltration footprint (either Line 21 or Line 23)
2. The DCV fraction of 0.375 is based on a 40% average annual percent capture and a 36-hour drawdown time.
3. The increase in footprint for volume reduction can be optimized using the approach presented in Appendix B.5.2. The optimized footprint cannot be smaller than the alternative minimum footprint sizing factor from Worksheet B.5-2.
4. If the proposed biofiltration BMP footprint is smaller than the alternative minimum footprint sizing factor from Worksheet B.5-2, but satisfies Option 1 or Option 2 sizing, it is considered a compact biofiltration BMP and may be allowed at the discretion of the City Engineer, if it meets the requirements in Appendix F.

BMP design follows the City of San Diego Storm Water Standards Manual. The BMP is sized using standard ensuring maximization of retention and pollutant removal.
This BMP is sized to have a minimum footprint of 3% of the contributing area adjusted by the runoff factor.
With the proposed BMP parameters the minimum footprint exceeds the footprint required to Biofilter 1.5 times the DCV. It also exceeds the footprint required to store 0.75 of the remaining DCV in pores and ponding



NOTE:
The 85th percentile is a 24 hour rainfall total. It represetns a value such that 85% of the observed 24 hour rainfall totals will be less than that value.

05/14/2015
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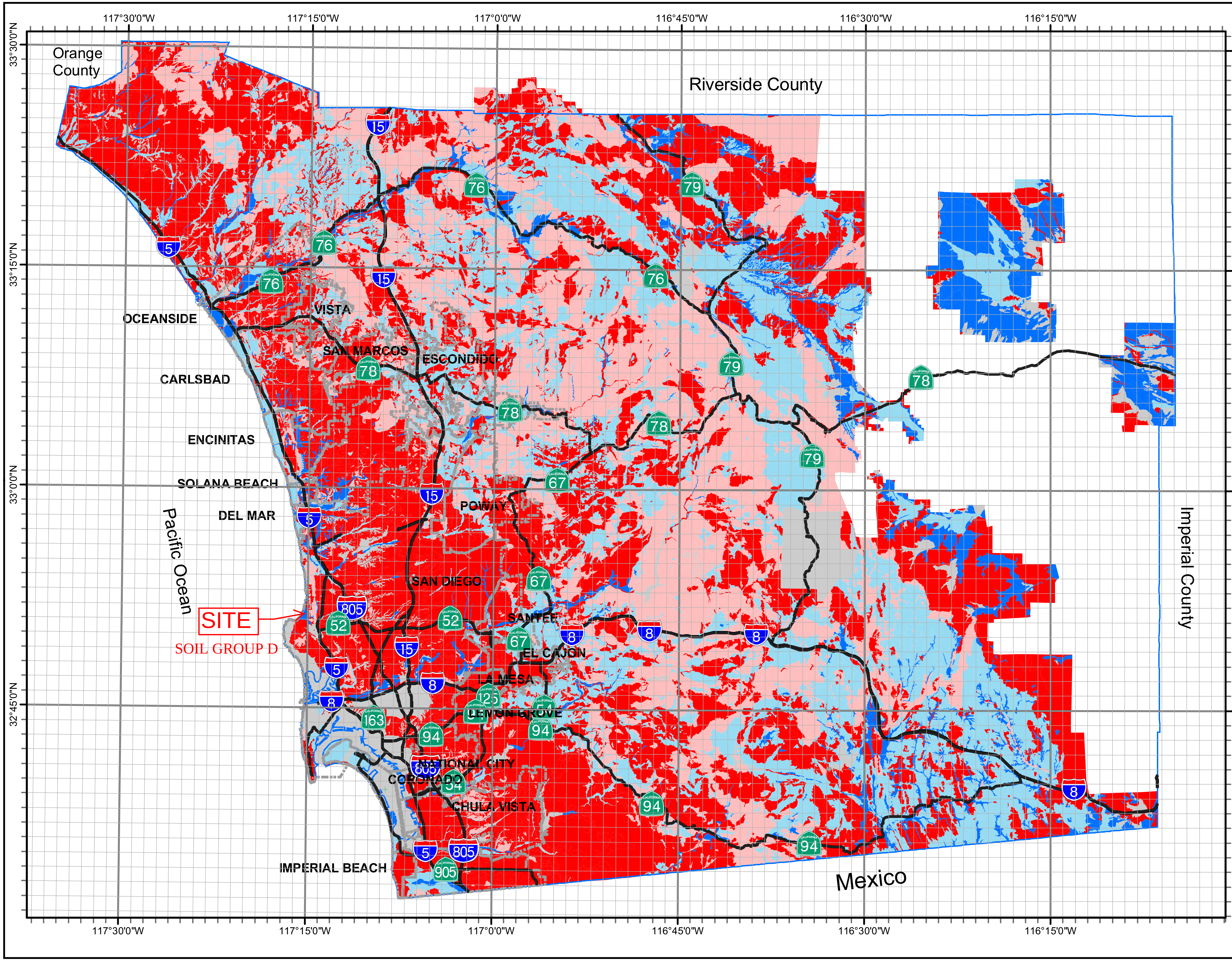
\PCT85_DISPLAY.mxd

Figure B.1-1: 85th Percentile 24-hour Isopluvial Map



County of San Diego Hydrology Manual Soil Hydrologic Group

Legend

- ▬ Major Roads
- ▭ Incorporated City Bdy

HYDROLOGIC SOIL GROUP

- ⬜ Hydrologic Group Undefined
- 🟦 Hydrologic Group A
- 🟦 Hydrologic Group B
- 🟥 Hydrologic Group C
- 🟥 Hydrologic Group D
- ☐ No Soil Data

Note: Soil Data Source
USDA/NRCS
SSURGO Soils 2007

N

3 1.5 0 3 Miles

DPW GIS

SanGIS

E.13. BF-1 Biofiltration

Location: 43rd Street and Logan Avenue, San Diego, California

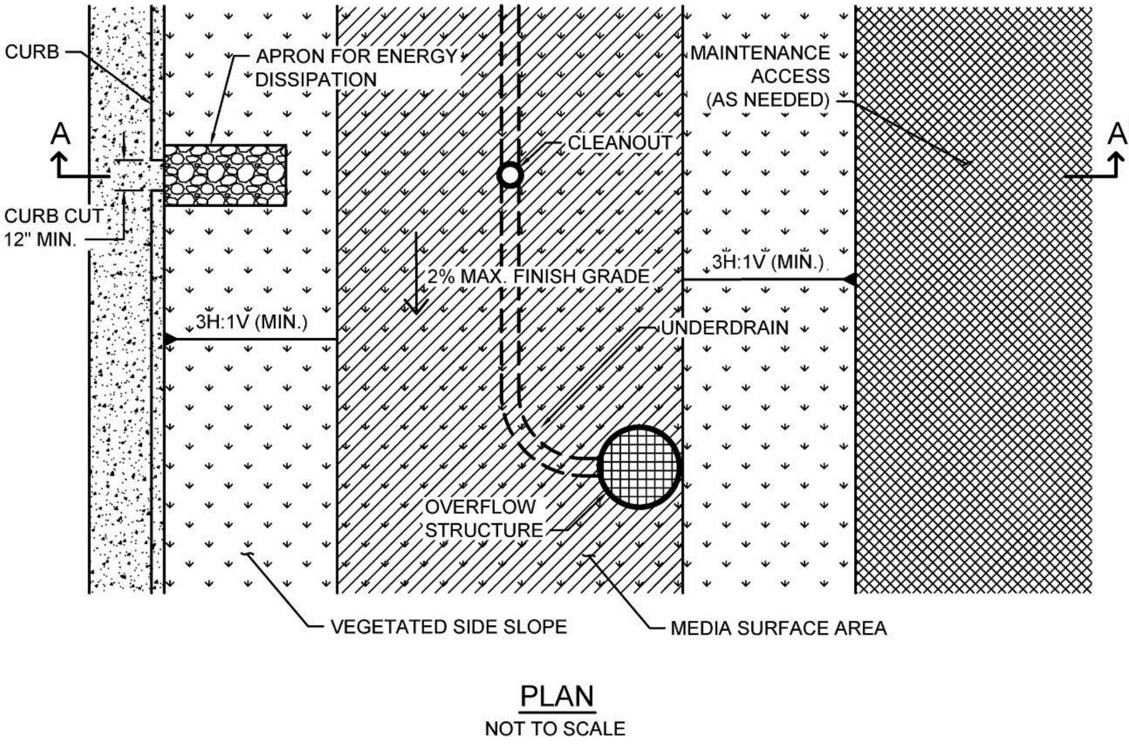
MS4 Permit Category
Biofiltration
Manual Category
Biofiltration
Applicable Performance Standard
Pollutant Control Flow Control
Primary Benefits
Treatment Volume Reduction (Incidental) Peak Flow Attenuation (Optional)

Description

Biofiltration (Bioretention with underdrain) facilities are vegetated surface water systems that filter water through vegetation, and soil or engineered media prior to discharge via underdrain or overflow to the downstream conveyance system. Bioretention with underdrain facilities are commonly incorporated into the site within parking lot landscaping, along roadsides, and in open spaces. Because these types of facilities have limited or no infiltration, they are typically designed to provide enough hydraulic head to move flows through the underdrain connection to the storm drain system. Treatment is achieved through filtration, sedimentation, sorption, biochemical processes and plant uptake.

Typical bioretention with underdrain components include:

- Inflow distribution mechanisms (e.g, perimeter flow spreader or filter strips)
- Energy dissipation mechanism for concentrated inflows (e.g., splash blocks or riprap)
- Shallow surface ponding for captured flows
- Side slope and basin bottom vegetation selected based on expected climate and ponding depth
- Non-floating mulch layer
- Media layer (planting mix or engineered media) capable of supporting vegetation growth
- Filter course layer (aka choking layer) consisting of aggregate to prevent the migration of fines into uncompacted native soils or the aggregate storage layer
- Aggregate storage layer with underdrain(s)
- Impermeable liner or uncompacted native soils at the bottom of the facility
- Overflow structure



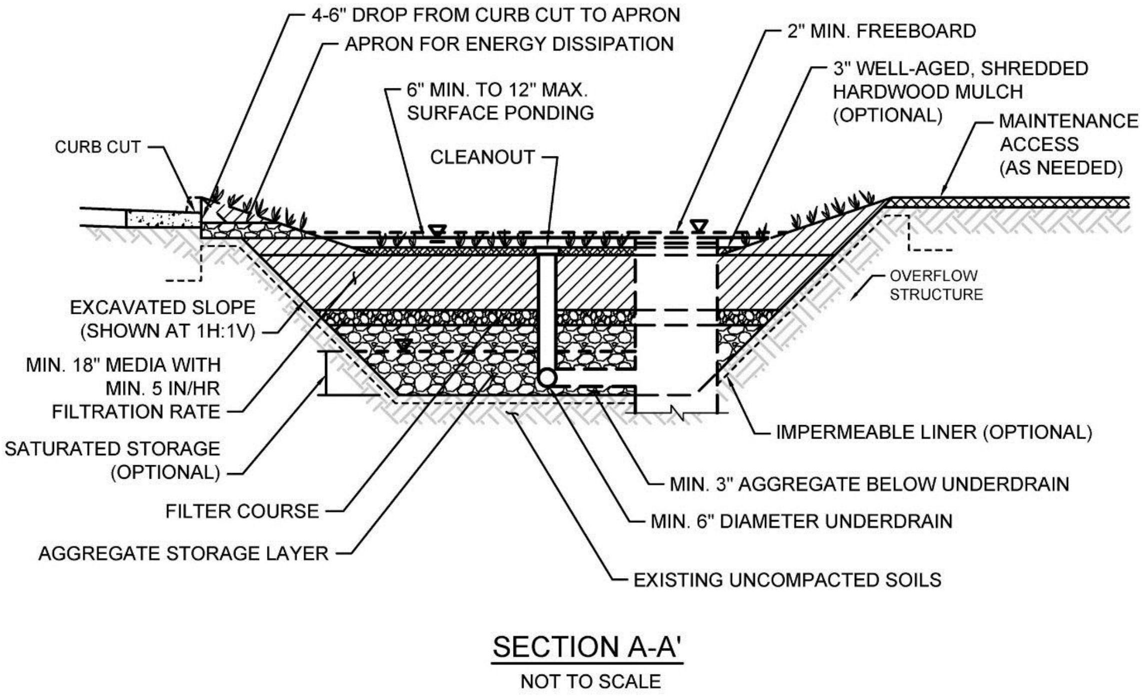


Figure E.13-E.13-1: Typical plan and Section view of a Biofiltration BMP

Design Adaptations for Project Goals

Biofiltration Treatment BMP for storm water pollutant control. The system is lined or un-lined to provide incidental infiltration, and an underdrain is provided at the bottom to carry away filtered runoff. This configuration is considered to provide biofiltration treatment via flow through the media layer. Storage provided above the underdrain within surface ponding, media, and aggregate storage is considered included in the biofiltration treatment volume. Saturated storage within the aggregate storage layer can be added to this design by raising the underdrain above the bottom of the aggregate storage layer or via an internal weir structure designed to maintain a specific water level elevation.

Integrated storm water flow control and pollutant control configuration. The system can be designed to provide flow rate and duration control by primarily providing increased surface ponding and/or having a deeper aggregate storage layer above the underdrain. This will allow for significant detention storage, which can be controlled via inclusion of an outlet structure at the downstream end of the underdrain.

Design Criteria and Considerations

Bioretention with underdrain must meet the following design criteria. Deviations from the below criteria may be approved at the discretion of the City Engineer if it is determined to be appropriate:

Siting and Design		Intent/Rationale
☑	Placement observes geotechnical recommendations regarding potential hazards (e.g., slope stability, landslides, liquefaction zones) and setbacks (e.g., slopes, foundations, utilities).	Must not negatively impact existing site geotechnical concerns.
☑	An impermeable liner or other hydraulic restriction layer is included if site constraints indicate that infiltration or lateral flows should not be allowed.	Lining prevents storm water from impacting groundwater and/or sensitive environmental or geotechnical features. Incidental infiltration, when allowable, can aid in pollutant removal and groundwater recharge.
☑	Contributing tributary area shall be ≤ 5 acres (≤ 1 acre preferred).	Bigger BMPs require additional design features for proper performance. Contributing tributary area greater than 5 acres may be allowed at the discretion of the City Engineer if the following conditions are met: 1) incorporate design features (e.g. flow spreaders) to minimizing short circuiting of flows in the BMP and 2) incorporate additional design features requested by the City Engineer for proper performance of the regional BMP.
☑	Finish grade of the facility is ≤ 2%.	Flatter surfaces reduce erosion and channelization within the facility.
Surface Ponding		

	Siting and Design	Intent/Rationale
☒	Surface ponding is limited to a 24-hour drawdown time.	Surface ponding limited to 24 hour for plant health. Surface ponding drawdown time greater than 24-hours but less than 96 hours may be allowed at the discretion of the City Engineer if certified by a landscape architect or agronomist.
☒	Surface ponding depth is ≥ 6 and ≤ 12 inches.	Surface ponding capacity lowers subsurface storage requirements. Deep surface ponding raises safety concerns. Surface ponding depth greater than 12 inches (for additional pollutant control or surface outlet structures or flow-control orifices) may be allowed at the discretion of the City Engineer if the following conditions are met: 1) surface ponding depth drawdown time is less than 24 hours; and 2) safety issues and fencing requirements are considered (typically ponding greater than 18" will require a fence and/or flatter side slopes) and 3) potential for elevated clogging risk is considered.
☒	A minimum of 2 inches of freeboard is provided.	Freeboard provides room for head over overflow structures and minimizes risk of uncontrolled surface discharge.
☒	Side slopes are stabilized with vegetation and are = 3H:1V or shallower.	Gentler side slopes are safer, less prone to erosion, able to establish vegetation more quickly and easier to maintain.
Vegetation		
☒	Plantings are suitable for the climate and expected ponding depth. A plant list to aid in selection can be found in Appendix E.20.	Plants suited to the climate and ponding depth are more likely to survive.
☒	An irrigation system with a connection to water supply should be provided as needed.	Seasonal irrigation might be needed to keep plants healthy.
Mulch (Mandatory)		
☒	A minimum of 3 inches of well-aged, shredded hardwood mulch that has been stockpiled or stored for at least 12 months is provided.	Mulch will suppress weeds and maintain moisture for plant growth. Aging mulch kills pathogens and weed seeds and allows the beneficial microbes to multiply.
Media Layer		

	Siting and Design	Intent/Rationale
☑	Media maintains a minimum filtration rate of 5 in/hr over lifetime of facility. Additional Criteria for media hydraulic conductivity described in the bioretention soil media model specification (Appendix F.4)	A filtration rate of at least 5 inches per hour allows soil to drain between events. The initial rate should be higher than long term target rate to account for clogging over time. However an excessively high initial rate can have a negative impact on treatment performance, therefore an upper limit is needed.
☑	Media is a minimum 18 inches deep, meeting the following media specifications: Model biorention soil media specification provided in Appendix F.4 <u>or</u> County of San Diego Low Impact Development Handbook: Appendix G - Bioretention Soil Specification (June 2014, unless superseded by more recent edition). Alternatively, for proprietary designs and custom media mixes not meeting the media specifications, the media meets the pollutant treatment performance criteria in Section F.1.	A deep media layer provides additional filtration and supports plants with deeper roots. Standard specifications shall be followed. For non-standard or proprietary designs, compliance with Appendix F.1 ensures that adequate treatment performance will be provided.
☑	Media surface area is 3% of contributing area times adjusted runoff factor or greater. Unless demonstrated that the BMP surface area can be smaller than 3%.	Greater surface area to tributary area ratios: a) maximizes volume retention as required by the MS4 Permit and b) decrease loading rates per square foot and therefore increase longevity. Adjusted runoff factor is to account for site design BMPs implemented upstream of the BMP (such as rain barrels, impervious area dispersion, etc.). Refer to Appendix B.2 guidance. Use Worksheet B.5-1 Line 26 to estimate the minimum surface area required per this criteria.
☐	Where receiving waters are impaired or have a TMDL for nutrients, the system is designed with nutrient sensitive media design (see fact sheet BF-2).	Potential for pollutant export is partly a function of media composition; media design must minimize potential for export of nutrients, particularly where receiving waters are impaired for nutrients.
Filter Course Layer		
☑	A filter course is used to prevent migration of fines through layers of the facility. Filter fabric is not used.	Migration of media can cause clogging of the aggregate storage layer void spaces or subgrade and can result in poor water quality performance for turbidity and suspended solids. Filter fabric is more likely to clog.

	Siting and Design	Intent/Rationale
☑	Filter course is washed and free of fines.	Washing aggregate will help eliminate fines that could clog the facility and impede infiltration.
☑	To reduce clogging potential, a two-layer filter course (aka choking stone system) is used consisting of one 3" layer of clean and washed ASTM 33 Fine Aggregate Sand overlying a 3" layer of ASTM No 8 Stone (Appendix F.5).	This specification has been developed to maintain permeability while limiting the migration of media material into the stone reservoir and underdrain system.
Aggregate Storage Layer		
☑	ASTM #57 open graded stone is used for the storage layer and a two layer filter course (detailed above) is used above this layer	This layer provides additional storage capacity. ASTM #8 stone provides an acceptable choking/bridging interface with the particles in ASTM #57 stone.
☑	The depth of aggregate provided (12-inch typical) and storage layer configuration is adequate for providing conveyance for underdrain flows to the outlet structure.	Proper storage layer configuration and underdrain placement will minimize facility drawdown time.
Inflow, Underdrain, and Outflow Structures		
☑	Inflow, underdrains and outflow structures are accessible for inspection and maintenance.	Maintenance will prevent clogging and ensure proper operation of the flow control structures.
☑	Inflow velocities are limited to 3 ft/s or less or use energy dissipation methods. (e.g., riprap, level spreader) for concentrated inflows.	High inflow velocities can cause erosion, scour and/or channeling.
☐	Curb cut inlets are at least 12 inches wide, have a 4-6 inch reveal (drop) and an apron and energy dissipation as needed.	Inlets must not restrict flow and apron prevents blockage from vegetation as it grows in. Energy dissipation prevents erosion.
☑	Underdrain outlet elevation should be a minimum of 3 inches above the bottom elevation of the aggregate storage layer.	A minimal separation from subgrade or the liner lessens the risk of fines entering the underdrain and can improve hydraulic performance by allowing perforations to remain unblocked.
☑	Minimum underdrain diameter is 8 inches.	Smaller diameter underdrains are prone to clogging.
☑	Underdrains should be affixed with an upturned elbow to an elevation at least 9 to 12 inches above the invert of the underdrain.	An upturned elbow reduces velocity in the underdrain pipe and can help reduce mobilization of sediments from the underdrain and media bed.

	Siting and Design	Intent/Rationale
☒	Underdrains are made of slotted, PVC pipe conforming to ASTM D 3034 or equivalent or corrugated, HDPE pipe conforming to AASHTO 252M or equivalent.	Slotted underdrains provide greater intake capacity, clog resistant drainage, and reduced entrance velocity into the pipe, thereby reducing the chances of solids migration.
☒	An underdrain cleanout with a minimum 8-inch diameter and lockable cap is placed every 50 feet as required based on underdrain length.	Properly spaced cleanouts will facilitate underdrain maintenance.
☒	Overflow is safely conveyed to a downstream storm drain system or discharge point Size overflow structure to pass 100-year peak flow for on-line infiltration basins and water quality peak flow for off-line basins.	Planning for overflow lessens the risk of property damage due to flooding.

Conceptual Design and Sizing Approach for Storm Water Pollutant Control Only

To design bioretention with underdrain for storm water pollutant control only (no flow control required), the following steps should be taken:

1. Verify that siting and design criteria have been met, including placement requirements, contributing tributary area, maximum side and finish grade slopes, and the recommended media surface area tributary ratio.
2. Calculate the DCV per Appendix B based on expected site design runoff for tributary areas.
3. Use the sizing worksheet presented in Appendix B.5 to size biofiltration BMPs.

Conceptual Design and Sizing Approach when Storm Water Flow Control is Applicable

Control of flow rates and/or durations will typically require significant surface ponding and/or aggregate storage volumes, and therefore the following steps should be taken prior to determination of storm water pollutant control design. Pre-development and allowable post-project flow rates and durations should be determined as discussed in Chapter 6 of the manual.

1. Verify that siting and design criteria have been met, including placement requirements, contributing tributary area, maximum side and finish grade slopes, and the recommended media surface area tributary ratio.
2. Iteratively determine the facility footprint area, surface ponding and/or aggregate storage layer depth required to provide detention storage to reduce flow rates and durations to allowable limits. Flow rates and durations can be controlled from detention storage by altering outlet structure orifice size(s) and/or water control levels. Multi-level orifices can be used within an outlet structure to control the full range of flows.
3. If bioretention with underdrain cannot fully provide the flow rate and duration control required by this manual, an upstream or downstream structure with significant storage volume such as an underground vault can be used to provide remaining controls.
4. After bioretention with underdrain has been designed to meet flow control requirements, calculations must be completed to verify if storm water pollutant control requirements to treat the DCV have been met.

1	**Biofiltration BMPs shall be allowed to be used only as described in the BMP selection process based on a documented feasibility analysis.** Intent: This manual defines a specific prioritization of pollutant treatment BMPs, where BMPs that retain water (retained includes evapotranspired, infiltrated, and/or harvested and used) must be used before considering BMPs that have a biofiltered discharge to the MS4 or surface waters. Use of a biofiltration BMP in a manner in conflict with this prioritization (i.e., without a feasibility analysis justifying its use) is not permitted, regardless of the adequacy of the sizing and design of the system.	
☒	The project applicant has demonstrated that it is not technically feasible to retain the full DCV onsite.	Document feasibility analysis and findings in SWQMP per Appendix C.
2	**Biofiltration BMPs must be sized using acceptable sizing methods.** Intent: The MS4 Permit and this manual defines specific sizing methods that must be used to size biofiltration BMPs. Sizing of biofiltration BMPs is a fundamental factor in the amount of storm water that can be treated and also influences volume and pollutant retention processes.	
☒	The project applicant has demonstrated that biofiltration BMPs are sized to meet one of the biofiltration sizing options available (Appendix B.5).	Submit sizing worksheets (Appendix B.5) or other equivalent documentation with the SWQMP.
3	**Biofiltration BMPs must be sited and designed to achieve maximum feasible infiltration and evapotranspiration.** Intent: Various decisions about BMP placement and design influence how much water is retained via infiltration and evapotranspiration. The MS4 Permit requires that biofiltration BMPs achieve maximum feasible retention (evapotranspiration and infiltration) of storm water volume.	
NO INFILTRATION ☒	The biofiltration BMP is sited to allow for maximum infiltration of runoff volume based on the feasibility factors considered in site planning efforts. It is also designed to maximize evapotranspiration through the use of amended media and plants (biofiltration designs without amended media and plants may be permissible; see Item 5).	Document site planning and feasibility analyses in SWQMP per Section 5.4.
N/A ☐	For biofiltration BMPs categorized as "Partial Infiltration Condition," the infiltration storage depth in the biofiltration design has been selected to drain in 36 hours (+/-25%) or an alternative value shown to maximize infiltration on the site.	Included documentation of estimated infiltration rate per Appendix D; provide calculations using Appendix B.4 and B.5 to show that the infiltration storage depth meets this criterion. Note, depths that are too shallow or too deep may not be acceptable.

N/A	☐	For biofiltration BMP locations categorized as “Partial Infiltration Condition,” the infiltration storage is over the entire bottom of the biofiltration BMP footprint.	Document on plans that the infiltration storage covers the entire bottom of the BMP (i.e., not just underdrain trenches); or an equivalent footprint elsewhere on the site.
N/A	☐	For biofiltration BMP locations categorized as “Partial Infiltration Condition,” the sizing factor used for the infiltration storage area is not less than the minimum biofiltration BMP sizing factors calculated using Worksheet B.5.1.	Provide a table that compares the minimum sizing factor per Worksheet B.5.1 to the provided sizing factor. Note: The infiltration storage area could be a separate storage feature located downstream of the biofiltration BMP, not necessarily within the same footprint.
	☒	An impermeable liner or other hydraulic restriction layer is only used when needed to avoid geotechnical and/or subsurface contamination issues in locations identified as “No Infiltration Condition.”	If using an impermeable liner or hydraulic restriction layer, provide documentation of feasibility findings per Appendix C that recommend the use of this feature.
N/A	☐	The use of “compact” biofiltration BMP design[8] is permitted only in conditions identified as “No Infiltration Condition” and where site-specific documentation demonstrates that the use of larger footprint biofiltration BMPs would be infeasible.	Provide documentation of feasibility findings that recommend no infiltration is feasible. Provide site-specific information to demonstrate that a larger footprint biofiltration BMP would not be feasible.
	4	**Biofiltration BMPs must be designed with a hydraulic loading rate to maximize pollutant retention, preserve pollutant control processes, and minimize potential for pollutant washout.** Intent: Various decisions about biofiltration BMP design influence the degree to which pollutants are retained. The MS4 Permit requires that biofiltration BMPs achieve maximum feasible retention of storm water pollutants.	

CHECKLIST TO BE COMPLETED IN FINAL
WHEN FINAL BIOFILTRATION DETAILS ARE SUBMITTED

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[8] Compact biofiltration BMPs are defined as features with infiltration storage footprint less than the minimum sizing factors required to achieve 40% volume retention. Note that if a biofiltration BMP is accompanied by an infiltrating area downstream that has a footprint equal to at least the minimum sizing factors calculated using Worksheet B.5.1 assuming a partial infiltration condition, then it is not considered to be a compact biofiltration BMP for the purpose of Item 4 of the checklist. For potential configurations with a higher rate biofiltration BMP upstream of an larger footprint infiltration area, the BMP would still need to comply with Item 5 of this checklist for pollutant treatment effectiveness.

☐	Media selected for the biofiltration BMP meets minimum quality and material specifications per Appendix F.4 or County LID Manual, including the maximum allowable design filtration rate and minimum thickness of media.	Provide documentation that media meets the specifications in Appendix F.4 or County LID Manual.
	OR	
☐	Alternatively, for proprietary designs and custom media mixes not meeting the media specifications contained in Appendix F.4 or County LID Manual, field scale testing data are provided to demonstrate that proposed media meets the pollutant treatment performance criteria in Section F.1 below.	Provide documentation of performance information as described in Section F.1.
☑	To the extent practicable, filtration rates are outlet controlled (e.g., via an underdrain and orifice/weir) instead of controlled by the infiltration rate of the media.	Include outlet control in designs or provide documentation of why outlet control is not practicable.
☑	The water surface drains to at least 12 inches below the media surface within 24 hours from the end of storm event flow to preserve plant health and promote healthy soil structure.	Include calculations to demonstrate that drawdown rate is adequate. Surface ponding drawdown time greater than 24-hours but less than 96 hours may be allowed at the discretion of the City Engineer if certified by a landscape architect or agronomist.
☐	If nutrients are a pollutant of concern, design of the biofiltration BMP follows nutrient-sensitive design criteria.	Follow specifications for nutrient sensitive design in Fact Sheet BF-2. Or provide alternative documentation that nutrient treatment is addressed and potential for nutrient release is minimized.
☐	Media gradation calculations demonstrate that migration of media between layers will be prevented and permeability will be preserved.	Follow specification for choking layer in Fact Sheet PR-1 or BF-1. Or include calculations to demonstrate that choking layer is appropriately specified.
5	**Biofiltration BMPs must be designed to promote appropriate biological activity to support and maintain treatment processes.** Intent: Biological processes are an important element of biofiltration performance and longevity.	

PLANTS TO BE CONFIRMED IN LANDSCAPE PLANS	☐	Plants have been selected to be tolerant of project climate, design ponding depths and the treatment media composition.	Provide documentation justifying plant selection. Refer to the plant list in Appendix E.20.
	☐	Plants have been selected to minimize irrigation requirements.	Provide documentation describing irrigation requirements for establishment and long term operation.
	☐	Plant location and growth will not impede expected long-term media filtration rates and will enhance long term infiltration rates to the extent possible.	Provide documentation justifying plant selection. Refer to the plant list in Appendix E.20.
N/A	☐	If plants are not part of the biofiltration design, other biological processes are supported as needed to sustain treatment processes (e.g., biofilm in a subsurface flow wetland).	For biofiltration designs without plants, describe the biological processes that will support effective treatment and how they will be sustained. Refer to Appendix F.3
	6	**Biofiltration BMPs must be designed with a hydraulic loading rate to prevent erosion, scour, and channeling within the BMP.** Intent: Erosion, scour, and/or channeling can disrupt treatment processes and reduce biofiltration effectiveness.	
	☒	Scour protection has been provided for both sheet flow and pipe inflows to the BMP, where needed.	Provide documentation of scour protection as described in Fact Sheets PR-1 or BF-1 or approved equivalent.
	☒	Where scour protection has not been provided, flows into and within the BMP are kept to non-erosive velocities.	Provide documentation of design checks for erosive velocities as described in Fact Sheets PR-1 or BF-1 or approved equivalent.
N/A	☐	For proprietary BMPs, the BMP is used in a manner consistent with manufacturer guidelines and conditions of its third-party certification[9] (i.e., maximum tributary area, maximum inflow velocities, etc., as applicable).	Provide copy of manufacturer recommendations and conditions of third-party certification.

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[9] Certifications or verifications issued by the Washington Technology Acceptance Protocol-Ecology program and the New Jersey Corporation for Advanced Technology programs are typically accompanied by a set of guidelines regarding appropriate design and maintenance conditions that would be consistent with the certification/verification

7	**Biofiltration BMP must include operations and maintenance design features and planning considerations for continued effectiveness of pollutant and flow control functions.** Intent: Biofiltration BMPs require regular maintenance in order provide ongoing function as intended. Additionally, it is not possible to foresee and avoid potential issues as part of design; therefore plans must be in place to correct issues if they arise.	
☐ FINAL O&M TO BE PROVIDED IN MINISTERIAL	The biofiltration BMP O&M plan describes specific inspection activities, regular/periodic maintenance activities and specific corrective actions relating to scour, erosion, channeling, media clogging, vegetation health, and inflow and outflow structures.	Include O&M plan with project submittal as described in Chapter 7.
☐	Adequate site area and features have been provided for BMP inspection and maintenance access.	Illustrate maintenance access routes, setbacks, maintenance features as needed on project water quality plans.
☐	For proprietary biofiltration BMPs, the BMP maintenance plan is consistent with manufacturer guidelines and conditions of its third-party certification (i.e., maintenance activities, frequencies).	Provide copy of manufacturer recommendations and conditions of third-party certification.

Project Name: Klein Residence

Use this checklist to ensure the required information has been included on the DMA Exhibit:

The DMA Exhibit must identify:

☒ Underlying hydrologic soil group
☒ Approximate depth to groundwater
☒ Existing natural hydrologic features (watercourses, seeps, springs, wetlands)
☒ Critical coarse sediment yield areas to be protected
☒ Existing topography and impervious areas
☒ Existing and proposed site drainage network and connections to drainage offsite
☒ Proposed grading
☒ Proposed impervious features
☒ Proposed design features and surface treatments used to minimize imperviousness
☒ Drainage management area (DMA) boundaries, DMA ID numbers, and DMA areas (square footage or acreage), and DMA type (i.e., drains to BMP, self-retaining, or self-mitigating)
☒ Potential pollutant source areas and corresponding required source controls (see Chapter 4, Appendix E.1, and Form I-3B)
☒ Structural BMPs (identify location, type of BMP, and size/detail)

ATTACHMENT 2
BACKUP FOR PDP HYDROMODIFICATION CONTROL MEASURES

This is the cover sheet for Attachment 2.

☒ Mark this box if this attachment is empty because the project is exempt from PDP hydromodification management requirements.

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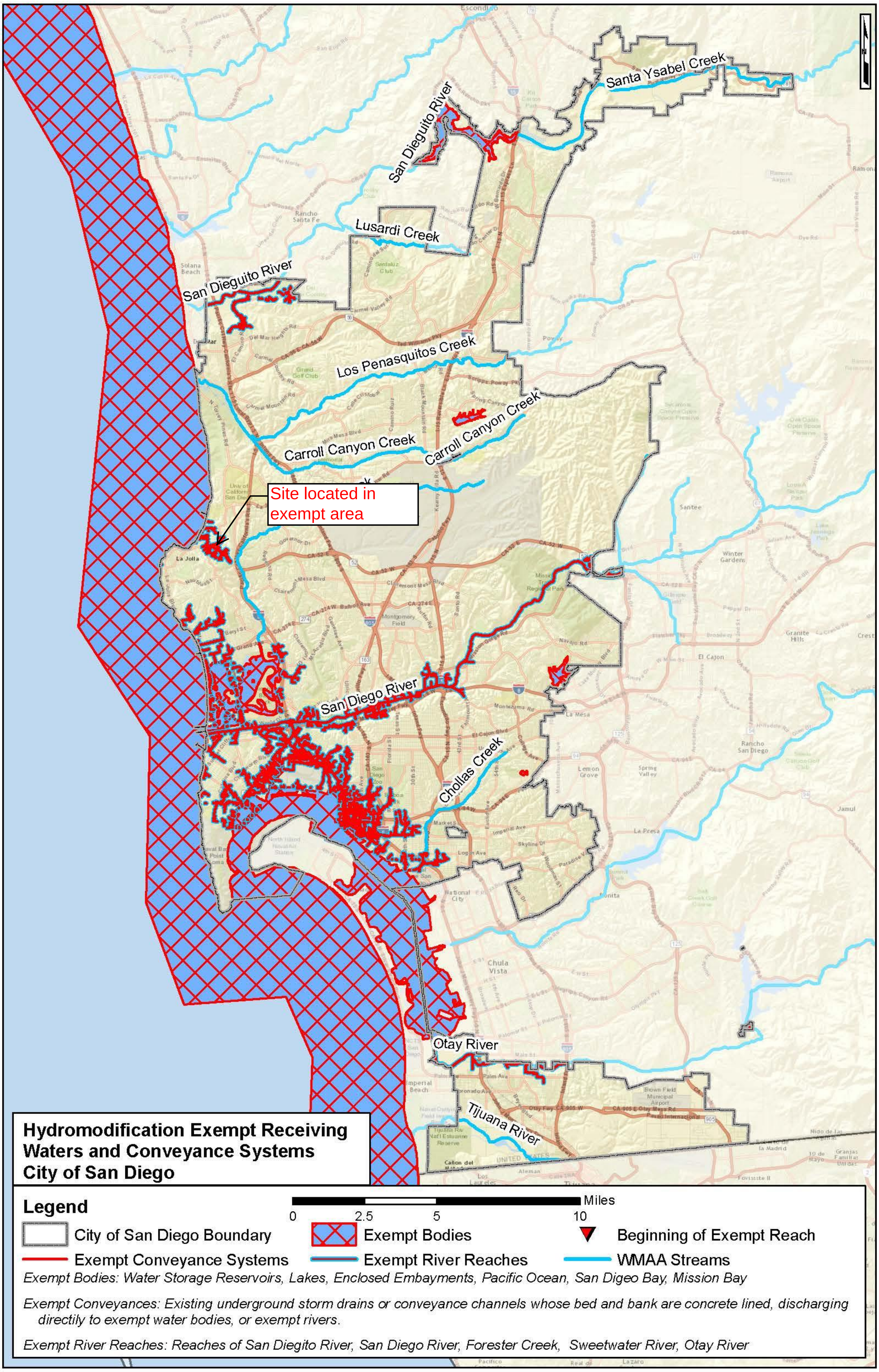
Project Name: Klein Residence

Indicate which Items are Included:

Attachment Sequence	Contents	Checklist
Attachment 2a	Hydromodification Management Exhibit (Required)	☐ Included See Hydromodification Management Exhibit Checklist.
Attachment 2b	Management of Critical Coarse Sediment Yield Areas (WMAA Exhibit is required, additional analyses are optional) See Section 6.2 of the BMP Design Manual.	☐ Exhibit showing project drainage boundaries marked on WMAA Critical Coarse Sediment Yield Area Map (Required) Optional analyses for Critical Coarse Sediment Yield Area Determination ☐ 6.2.1 Verification of Geomorphic Landscape Units Onsite ☐ 6.2.2 Downstream Systems Sensitivity to Coarse Sediment ☐ 6.2.3 Optional Additional Analysis of Potential Critical Coarse Sediment Yield Areas Onsite
Attachment 2c	Geomorphic Assessment of Receiving Channels (Optional) See Section 6.3.4 of the BMP Design Manual.	○ Not Performed ○ Included ● Submitted as separate stand-alone document
Attachment 2d	Flow Control Facility Design and Structural BMP Drawdown Calculations (Required) Overflow Design Summary for each structural BMP See Chapter 6 and Appendix G of the BMP Design Manual	○ Included ● Submitted as separate stand-alone document
Attachment 2e	Vector Control Plan (Required when structural BMPs will not drain in 96 hours)	○ Included ● Not required because BMPs will drain in less than 96 hours



Exempt Bodies: Water Storage Reservoirs, Lakes, Enclosed Embayments, Pacific Ocean, San Digeo Bay, Mission Bay

Exempt Conveyances: Existing underground storm drains or conveyance channels whose bed and bank are concrete lined, discharging directly to exempt water bodies, or exempt rivers.

Exempt River Reaches: Reaches of San Diegito River, San Diego River, Forester Creek, Sweetwater River, Otay River

Figure H-G.2-2 Hydromodification Exempt Areas

Use this checklist to ensure the required information has been included on the Hydromodification Management Exhibit:

The Hydromodification Management Exhibit must identify:

☐ Underlying hydrologic soil group
☐ Approximate depth to groundwater
☐ Existing natural hydrologic features (watercourses, seeps, springs, wetlands)
☐ Critical coarse sediment yield areas to be protected
☐ Existing topography
☐ Existing and proposed site drainage network and connections to drainage offsite
☐ Proposed grading
☐ Proposed impervious features
☐ Proposed design features and surface treatments used to minimize imperviousness
☐ Point(s) of Compliance (POC) for Hydromodification Management
☐ Existing and proposed drainage boundary and drainage area to each POC (when necessary, create separate exhibits for pre-development and post-project conditions)
☐ Structural BMPs for hydromodification management (identify location, type of BMP, and size/detail)

ATTACHMENT 3
STRUCTURAL BMP MAINTENANCE INFORMATION

This is the cover sheet for Attachment 3.

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Project Name: **Klein Residence**

Indicate which Items are Included:

Attachment Sequence	Contents	Checklist
Attachment 3a	Structural BMP Maintenance Thresholds and Actions (Required)	☒ Included See Structural BMP Maintenance Information Checklist.
Attachment 3b	Maintenance Agreement (Form DS-3247) (when applicable)	○ Included ● Not Applicable

Table 7-2. Maintenance Indicators and Actions for Vegetated BMPs

Typical Maintenance Indicator(s) for Vegetated BMPs	Maintenance Actions
Accumulation of sediment, litter, or debris	Remove and properly dispose of accumulated materials, without damage to the vegetation.
Poor vegetation establishment	Re-seed, re-plant, or re-establish vegetation per original plans.
Overgrown vegetation	Mow or trim as appropriate, but not less than the design height of the vegetation per original plans when applicable (e.g. a vegetated swale may require a minimum vegetation height).
Erosion due to concentrated irrigation flow	Repair/re-seed/re-plant eroded areas and adjust the irrigation system.
Erosion due to concentrated storm water runoff flow	Repair/re-seed/re-plant eroded areas, and make appropriate corrective measures such as adding erosion control blankets, adding stone at flow entry points, or minor re-grading to restore proper drainage according to the original plan. If the issue is not corrected by restoring the BMP to the original plan and grade, the City Engineer shall be contacted prior to any additional repairs or reconstruction.
Standing water in vegetated swales	Make appropriate corrective measures such as adjusting irrigation system, removing obstructions of debris or invasive vegetation, loosening or replacing top soil to allow for better infiltration, or minor re-grading for proper drainage. If the issue is not corrected by restoring the BMP to the original plan and grade, the City Engineer shall be contacted prior to any additional repairs or reconstruction.
Standing water in bioretention, biofiltration with partial retention, or biofiltration areas, or flow-through planter boxes for longer than 96 hours following a storm event*	Make appropriate corrective measures such as adjusting irrigation system, removing obstructions of debris or invasive vegetation, clearing underdrains (where applicable), or repairing/replacing clogged or compacted soils.
Obstructed inlet or outlet structure	Clear obstructions.
Damage to structural components such as weirs, inlet or outlet structures	Repair or replace as applicable.
*These BMPs typically include a surface ponding layer as part of their function which may take 96 hours to drain following a storm event.	

Project Name: Klein Residence

Use this checklist to ensure the required information has been included in the Structural BMP Maintenance Information Attachment:

Preliminary Design / Planning / CEQA level submittal:

- Attachment 3a must identify:
  - ☒ Typical maintenance indicators and actions for proposed structural BMP(s) based on Section 7.7 of the BMP Design Manual
- Attachment 3b is not required for preliminary design / planning / CEQA level submittal.

Final Design level submittal:

Attachment 3a must identify:

☐ Specific maintenance indicators and actions for proposed structural BMP(s). This shall be based on Section 7.7 of the BMP Design Manual and enhanced to reflect actual proposed components of the structural BMP(s)
☐ How to access the structural BMP(s) to inspect and perform maintenance
☐ Features that are provided to facilitate inspection (e.g., observation ports, cleanouts, silt posts, or other features that allow the inspector to view necessary components of the structural BMP and compare to maintenance thresholds)
☐ Manufacturer and part number for proprietary parts of structural BMP(s) when applicable
☐ Maintenance thresholds specific to the structural BMP(s), with a location-specific frame of reference (e.g., level of accumulated materials that triggers removal of the materials, to be identified based on viewing marks on silt posts or measured with a survey rod with respect to a fixed benchmark within the BMP)
☐ When applicable, frequency of bioretention soil media replacement
☐ Recommended equipment to perform maintenance
☐ When applicable, necessary special training or certification requirements for inspection and maintenance personnel such as confined space entry or hazardous waste management

Attachment 3b: For private entity operation and maintenance, Attachment 3b must include a Storm Water Management and Discharge Control Maintenance Agreement (Form DS-3247). The following information must be included in the exhibits attached to the maintenance agreement:

☐ Vicinity map
☐ Site design BMPs for which DCV reduction is claimed for meeting the pollutant control obligations.
☐ BMP and HMP location and dimensions
☐ BMP and HMP specifications/cross section/model
☐ Maintenance recommendations and frequency
☐ LID features such as (permeable paver and LS location, dim, SF).

NOW, THEREFORE, the parties agree as follows:

1. Property Owner shall have prepared, or if qualified, shall prepare an Operation and Maintenance Procedure [OMP] for Permanent Storm Water BMP's, satisfactory to the City, according to the attached exhibit(s), consistent with the Grading and/or Improvement Plan Drawing No(s), or Building Plan Project No(s):Click or tap here to enter text..

2. Property Owner shall install, maintain and repair or replace all Permanent Storm Water BMP's within their property, according to the OMP guidelines as described in the attached exhibit(s), the project's WQTR and Grading and/or Improvement Plan Drawing No(s), or Building Plan Project No(s)Click or tap here to enter text..

3. Property Owner shall maintain operation and maintenance records for at least five (5) years. These records shall be made available to the City for inspection upon request at any time.

This Maintenance Agreement shall commence upon execution of this document by all parties named hereon, and shall run with the land.

Executed by the City of San Diego and by Property Owner in San Diego, California.

See Attached Exhibits(s):Click or tap here to enter text.

(Owner Signature)

Click or tap here to enter text.

(Print Name and Title)

Click or tap here to enter text.

(Company/Organization Name)

Click or tap to enter a date.

(Date)

THE CITY OF SAN DIEGO

APPROVED:

(City Control engineer Signature

(Print Name)

(Date)

NOTE: ALL SIGNATURES MUST INCLUDE NOTARY ACKNOWLEDMENTS PER CIVIL CODE SEC. 1180 ET.SEQ

ATTACHMENT 4
COPY OF PLAN SHEETS SHOWING PERMANENT STORM WATER BMPS

This is the cover sheet for Attachment 4.

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Use this checklist to ensure the required information has been included on the plans:

The plans must identify:

☒ Structural BMP(s) with ID numbers matching Form I-6 Summary of PDP Structural BMPs
☒ The grading and drainage design shown on the plans must be consistent with the delineation of DMAs shown on the DMA exhibit
☒ Details and specifications for construction of structural BMP(s)
☒ Signage indicating the location and boundary of structural BMP(s) as required by the City Engineer
☒ How to access the structural BMP(s) to inspect and perform maintenance
☒ Features that are provided to facilitate inspection (e.g., observation ports, cleanouts, silt posts, or other features that allow the inspector to view necessary components of the structural BMP and compare to maintenance thresholds)
☒ Manufacturer and part number for proprietary parts of structural BMP(s) when applicable
☒ Maintenance thresholds specific to the structural BMP(s), with a location-specific frame of reference (e.g., level of accumulated materials that triggers removal of the materials, to be identified based on viewing marks on silt posts or measured with a survey rod with respect to a fixed benchmark within the BMP)
☒ Recommended equipment to perform maintenance
☒ When applicable, necessary special training or certification requirements for inspection and maintenance personnel such as confined space entry or hazardous waste management
☒ Include landscaping plan sheets showing vegetation requirements for vegetated structural BMP(s)
☒ All BMPs must be fully dimensioned on the plans
☒ When propritery BMPs are used, site specific cross section with outflow, inflow and model number shall be provided. Broucher photocopies are not allowed.

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HYDROLOGY
and
HYDRAULICS REPORT

for
The Klein Residence

Prepared for:
City of San Diego
Development Services Department
1222 First Ave. MS 501
San Diego, CA 92101-4155

Project Site Address
2585 Calle Del Oro
La Jolla, CA 92037

Study Prepared by:
Omega Engineering Consultants
4340 Viewridge Ave Suite B
San Diego, CA 92123
(858) 634-8620

Preparation Date
December 7th, 2016

Andrew J. Kann	**RCE 50940**
Registration Expires	**9-30-2017**

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF APPENDICES

SITE AND PROJECT DESCRIPTION

This Hydrology and Hydraulics report has been prepared as part of the Grading and Development Plan set for the project located at 2585 Calle Del Oro. The project site is currently occupied by a 3,500 sq. ft. single family residence and proposes demolishing the existing structure and building a new single family residence. The proposed design implements a drainage system that conveys runoff to bioretention area and then a pump vault. The pump system will convey stormwater via force main system to the curb along Calle del Oro

The site drainage basin is located in the Scripps Hydrologic Area of the Peñasquitos Hydrologic Unit of the San Diego Hydraulic Region (906.30).See Figure No. 1 for the vicinity map. See Figure No. 2 for the existing drainage limits. See Figure No. 3 for the proposed drainage limits.

METHODOLOGY

This drainage report has been prepared in accordance with current City of San Diego regulations and procedures. All of the proposed pipes and facilities have been designed to intercept and convey the 100-year storm. The Modified Rational Method was used to compute the anticipated runoff. See the attached calculations for particulars. The following references have been used in preparation of this report:

(1) City of San Diego Hydrology Manual, April, 1984.
(2) Handbook of Hydraulics, E.F. Brater & H.W. King, 6th Ed., 1976.
(3) Modern Sewer Design, American Iron & Steel Institute, 1st Ed., 1980.
(4) County of San Diego Hydrology Manual, June, 2003

RATIONAL METHOD

Q=CIA

Where:

Q = peak discharge, in cubic feet per second (cfs)
C = runoff coefficient, proportion of the rainfall that runs off the surface (no units)
= $(0.90*(\% \text{ impervious}) + C_p*(1-\% \text{ Impervious}))$
I = average rainfall intensity for a duration equal to the Tc for the area, (in/hr)
= $7.44*P_6*T_c^{-0.645}$
A = drainage area contributing to the design location, in acres
C_p = Pervious Coefficient Runoff Value, from County Hydrology Manual Appendix A
= 0.35 for Type D soil
$T_c = \dfrac{1.8\ (1.1-C)*(T_c)^{0.5}}{S^{0.33}}$
S = Slope of drainage course

EXISTING SITE CONDITIONS:

The existing site consists of a single family residence located on a site that is approximately 0.45 acres and 46% impervious, with landscaped areas around the periphery. Existing drainage is facilitated via overland flow. The property is divided into two drainage basins EX-1 & EX-2. The westerly basin EX-1 flows west via surface flow to the neighbor's lot and eventually being intercepted by a curb inlet along Valecitos Rd. Flow from the easterly basin flows northeast to the gutter of Calle Del Oro. From there, it flows in along the street in a westerly direction until it is intercepted by a gutter at the corner of Calle del Cielo and Calle del Oro.

DEVELOPED SITE CONDITIONS:

The project proposes the removal of the existing structure and hardscape and the construction of a new multistory single family home in its place. The proposed building foot print will be 5,100 sf. The proposed site will be 51% impervious. Onsite drainage patterns will be modified due to surface modifications. A new storm drain system shall convey the majority of onsite runoff a pump vault in the back yard. The pump system will convey the runoff via force main to the gutter along Calle del Oro. Runoff that is pumped to the curb will follow the existing offsite flow path the curb inlet at the several hundred yards north of the site on Calle Del Oro. Drainage from a small portion of the site below the retaining wall be allowed to continue draining to the neighboring lot.

EXISTING RUNOFF ANALYSIS:

The Rational Method was used for calculating existing peak flow rates for the 85th %, and 100-year storms. Analysis of the existing conditions breaks the disturbed area into two separate drainage areas each with a separate discharge point. Runoff coefficients in the range of 0.43-0.85 were used for the existing basins.

DEVELOPED RUNOFF ANALYSIS:

The Rational Method was used for calculating proposed peak flow rates for the 85th% and 100-year storms. Analysis of the proposed site breaks the disturbed area into two separate drainage areas. The westerly basin (A-1) will surface flow to the existing discharge point along the westerly boundary of the site. The easterly basin (B-1) will discharge to the curb via a force main systems. Runoff coefficients in the range of 0.35-0.69 were used for the proposed basins.

RESULTS AND CONCLUSIONS

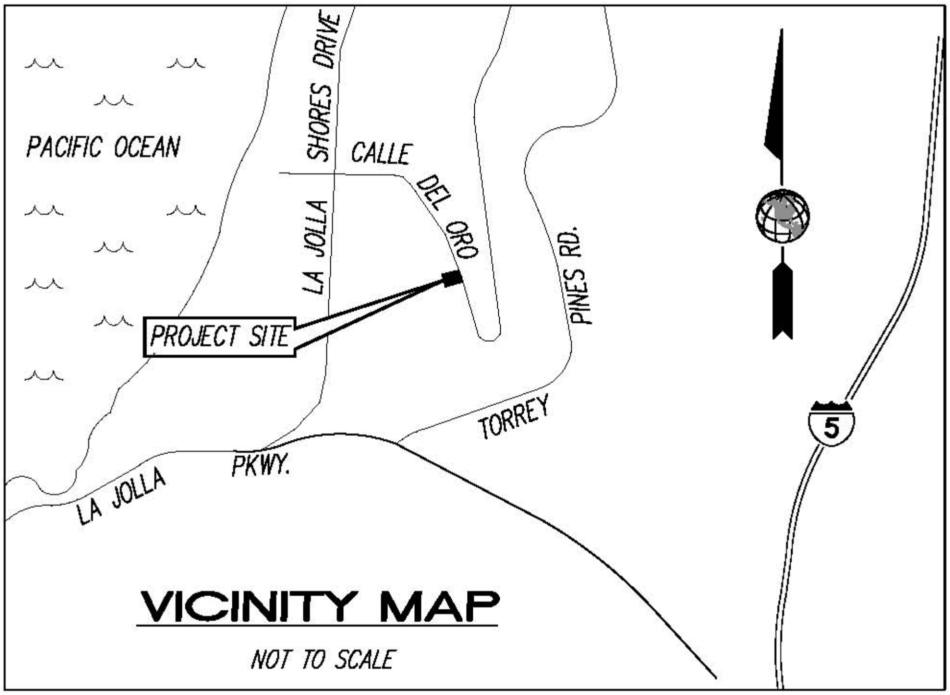
The redevelopment of the site will alter onsite drainage patterns by nearly eliminating overland flow to the downhill neighboring lots. Runoff will instead by discharge to the curb Calle del Oro by a forcemain pump system. The project will increase the amount of stormwater generated by the site, but the actual discharge rates from the site will be reduced. This is because the force main pumps will be limited to a constant outflow rate that is less than the existing discharge at Discharge Pt. #2. The pump vaults have been sized to store stormwater in excess of the discharge capacities of the pumps during the peak of high intensity storm events

The project will decrease the flow to the downhill areas west of the site and decrease the discharge to the curb from 0.81 to 0.38 cfs.

No CWA 401 or 404 permit is required as the project will not discharge fill or dredge material to a water of the state or US. The site will not dredge or place any fill in a water of the state or US.

Site design complies with ASBS requirements by draining all low flow runoff to biofiltration area. See separate SWQMP.

It is the opinion of Omega Engineering Consultants that this project will not negatively effect the downstream waterways and receiving water bodies.

PACIFIC OCEAN
LA JOLLA SHORES DRIVE
CALLE DEL ORO
PROJECT SITE
PINES RD.
TORREY
LA JOLLA PKWY.
5
VICINITY MAP
NOT TO SCALE

Klein Residence

HYDROLOGY AND HYDRAULICS CALCS (Table No. 1)

12/7/2016

BASIN	AREA (SF)	AREA (AC)	% Imp	"C" Value
EX-1	11,539	0.26	15.0%	0.43
EX-2	7,950	0.18	90.0%	0.85
EX. TOTAL	**19,489**	**0.45**		
A-1	3,317	0.08	0.0%	0.35
B-1	16,809	0.39	61.1%	0.69
PROP TOTAL	**20,126**	**0.46**		

Basin Confluence	Symbol
-	-
-	-

(A) "CP#1" Confluence Point Number 1

(B) C value for bare ground is 0.35 (Table 3-1 County Hydrology Manual)
C value for impervious surfaces is 0.9
Basins with mixed surface type use a weighted average
of these 2 values. (impervious % x 0.9)+(pervious % x 0.35)

Ex. and proposed areas totals are different as the exisiting and proposed sites have pools of different sizes

Klein Residence

HYDROLOGY AND HYDRAULICS CALCS (Table No. 2)

12/7/2016

Sub-Basin	AREA Ac.	"C"	CA	L (ft) Travel	H (ft) (elev)	S(%) (avg.)	Tc min.	T tot mins	I in/hr	Q cfs	Q tot cfs	L (ft) (Pipe)	S (%) (Pipe)	Dia. (in)	K'	D\d	pipe #	NOTES 85th % storm
EX-1	0.26	0.43	0.11	150	40.00	26.67	5.0	5.00	0.20	0.02	0.02							
							Discharge Pt #1 ex. discharge=				**0.02**	**CFS**						
EX-2	0.18	0.85	0.15	150	6.00	4.00	5.0	5.00	0.20	0.03	0.03							
							Discharge Pt #2 ex. discharge=				**0.03**	**CFS**						
A-1	0.08	0.35	0.03	15	6.00	40.00	5.0	5.00	0.20	0.01	0.01							
							Discharge Pt #1 prop. discharge=				**0.01**	**CFS**						
B-1	0.39	0.69	0.26	70	5.00	6.00	5.0	5.00	0.20	0.05	0.05							
									Pump Discharge=		0.43							
							Discharge Pt #1 prop. discharge=				**0.43**	**CFS**						

Klein Residence

HYDROLOGY AND HYDRAULICS CALCS (Table No. 2)

12/7/2016

Sub-Basin	AREA Ac.	"C"	CA	L (ft) Travel	H (ft) (elev)	S(%) (avg.)	Tc min.	T tot mins	I in/hr	Q cfs	Q tot cfs	L (ft) (Pipe)	S (%) (Pipe)	Dia. (in)	K'	D\d	pipe #	NOTES 100-yr
																	P(6)=	**2.00**
EX-1	0.26	0.43	0.11	150	40.00	26.67	5.0	5.00	5.27	0.60	0.60							
							Discharge Pt #1 ex. discharge=				**0.60**	**CFS**						
EX-2	0.18	0.85	0.15	150	6.00	4.00	5.0	5.00	5.27	0.81	0.81							
							Discharge Pt #2 ex. discharge=				**0.81**	**CFS**						
A-1	0.08	0.35	0.03	15	6.00	40.00	5.0	5.00	5.27	0.14	0.14							
							Discharge Pt #1 prop. discharge=				**0.14**	**CFS**						
B-1	0.39	0.69	0.26	70	5.00	6.00	5.0	5.00	5.27	1.40	1.40							
									Pump Discharge=		0.38							
							Discharge Pt #1 prop. discharge=				**0.38**	**CFS**						

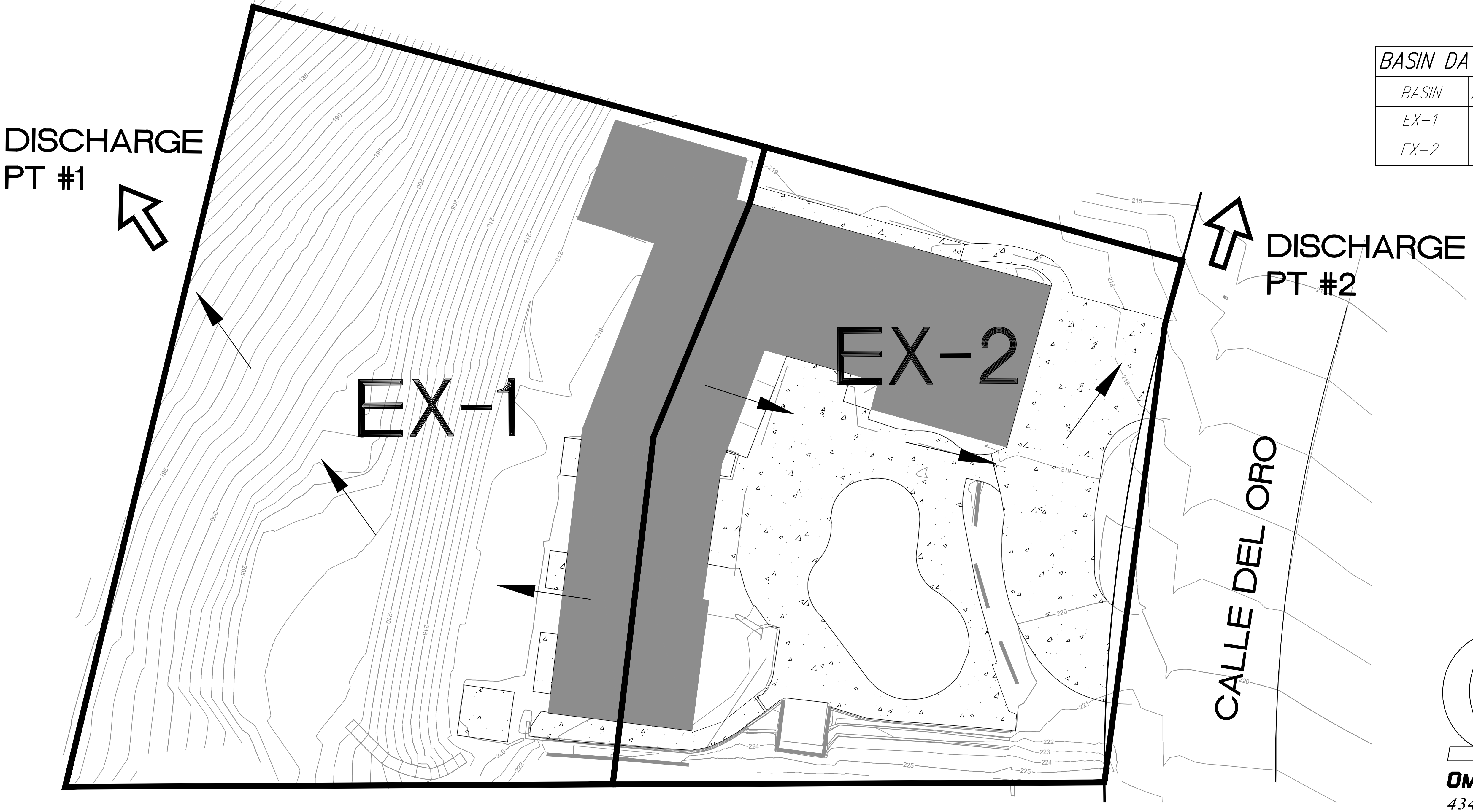
EXISTING HYDROLOGY EXHIBIT
THE KLEIN RESIDENCE

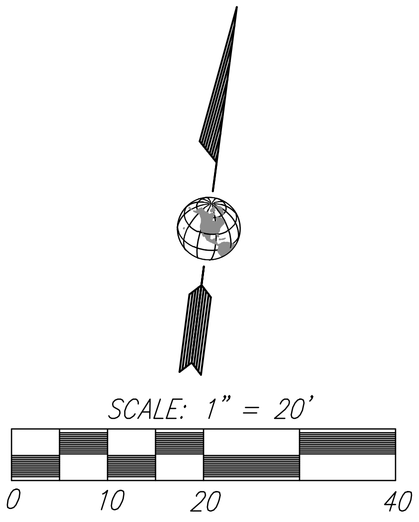
LEGEND:

AREA LIMITS	━━━
DRAINAGE DIRECTION ARROW	→ → →
BASIN NUMBER	A-#.#
BUILDING AREA	■
PAVEMENT AREA	░
PERVIOUS AREA	□

BASIN DATA

BASIN	*AREA (ac)*	*IMPERVIOUS %*	Q_{100}*(cfs)*
EX-1	*0.26*	*15.3%*	*0.60*
EX-2	*0.18*	*89.7%*	*0.81*





OMEGA ENGINEERING CONSULTANTS
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PH:(858) 634-8620 FAX:(858) 634-8627

PROPOSED HYDROLOGY EXHIBIT
THE KLEIN RESIDENCE

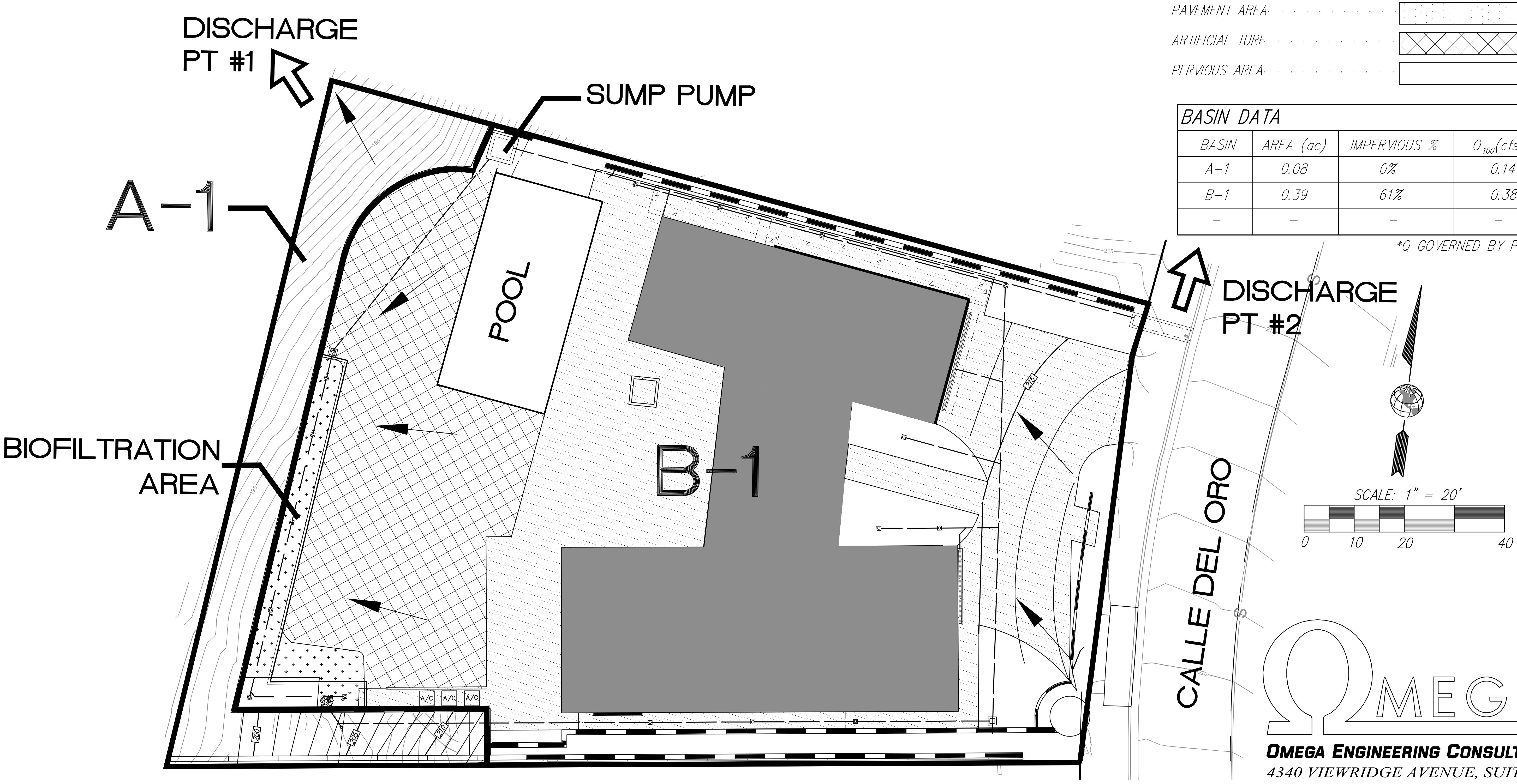
LEGEND:

AREA LIMITS	━━━━
DRAINAGE DIRECTION ARROW	→ → →
BASIN NUMBER	A-#
BUILDING AREA	■
PAVEMENT AREA	░
ARTIFICIAL TURF	▨
PERVIOUS AREA	☐

BASIN DATA

BASIN	AREA (ac)	IMPERVIOUS %	Q_{100}(cfs)
A-1	0.08	0%	0.14
B-1	0.39	61%	0.38*
-	-	-	-

*Q GOVERNED BY PUMP



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Basin B-1, force main head calculation

Total length = Pipe length + fitting equivalent lengths

<u>Pipe Length Lengths</u>

L= 135

<u>Equivalent Lengths</u>

2- 90^o elbows @ 5.0 ft ea=	10
1- Ball Valve @ 1.2 ft ea=	1.2
Total=	11.2

<u>Total Length</u>

146.2

<u>Flowrate (Q)</u>

0.38	CFS
170.544	GPM
170.544	GPM per pipe

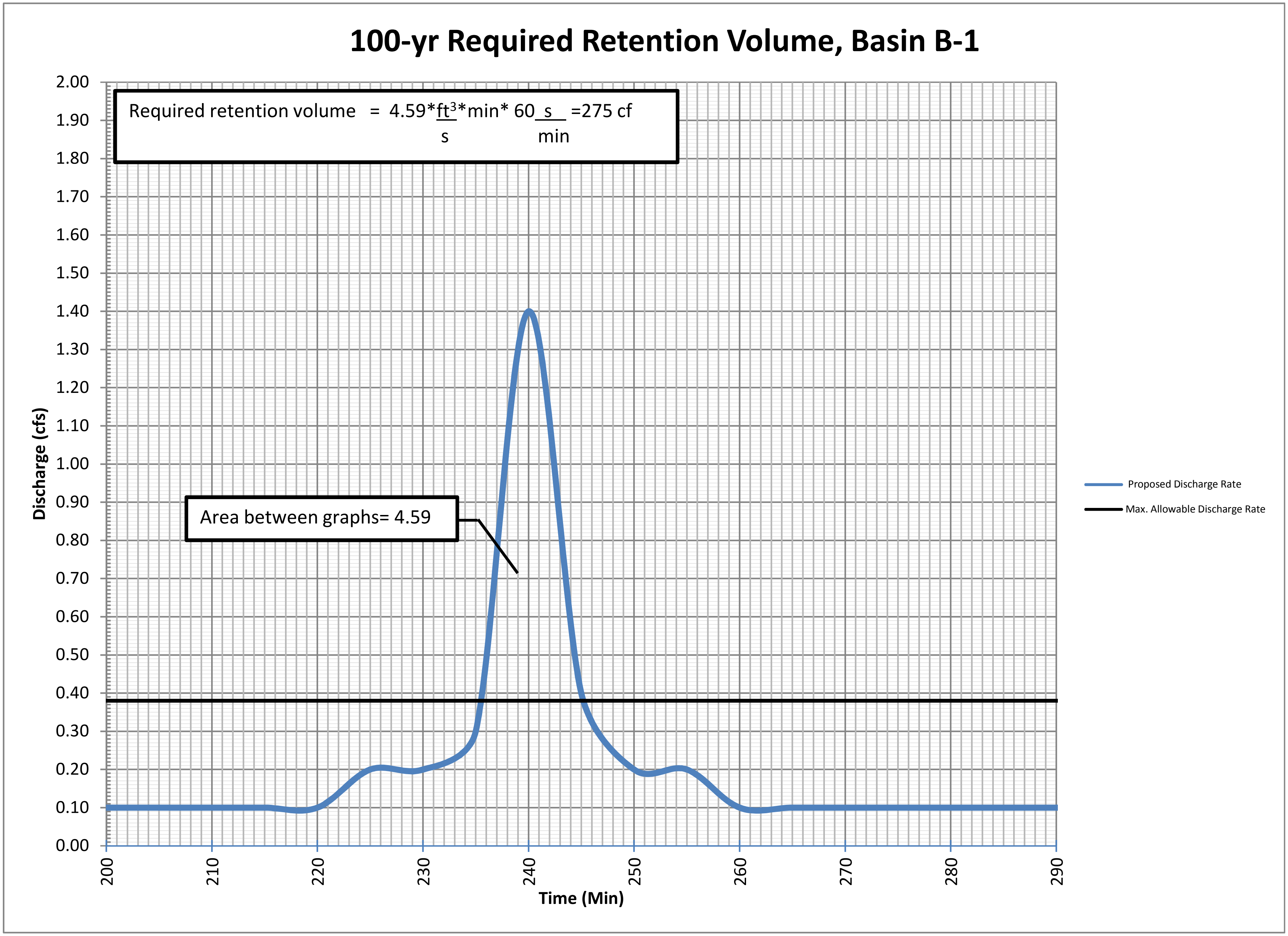
<u>Total Head = H_{static} + $H_{friction}$</u>

H_{static} = 28 ft

$$H_{friction} = 0.2083 \times \left(\frac{100}{C}\right)^{1.852} \times \left(\frac{Q^{1.852}}{D_h{}^{4.8655}}\right) \times \frac{L}{100} = \quad 9.32$$

C=150 Q=90 gpm D_h=3.0

Total Head = 37.32



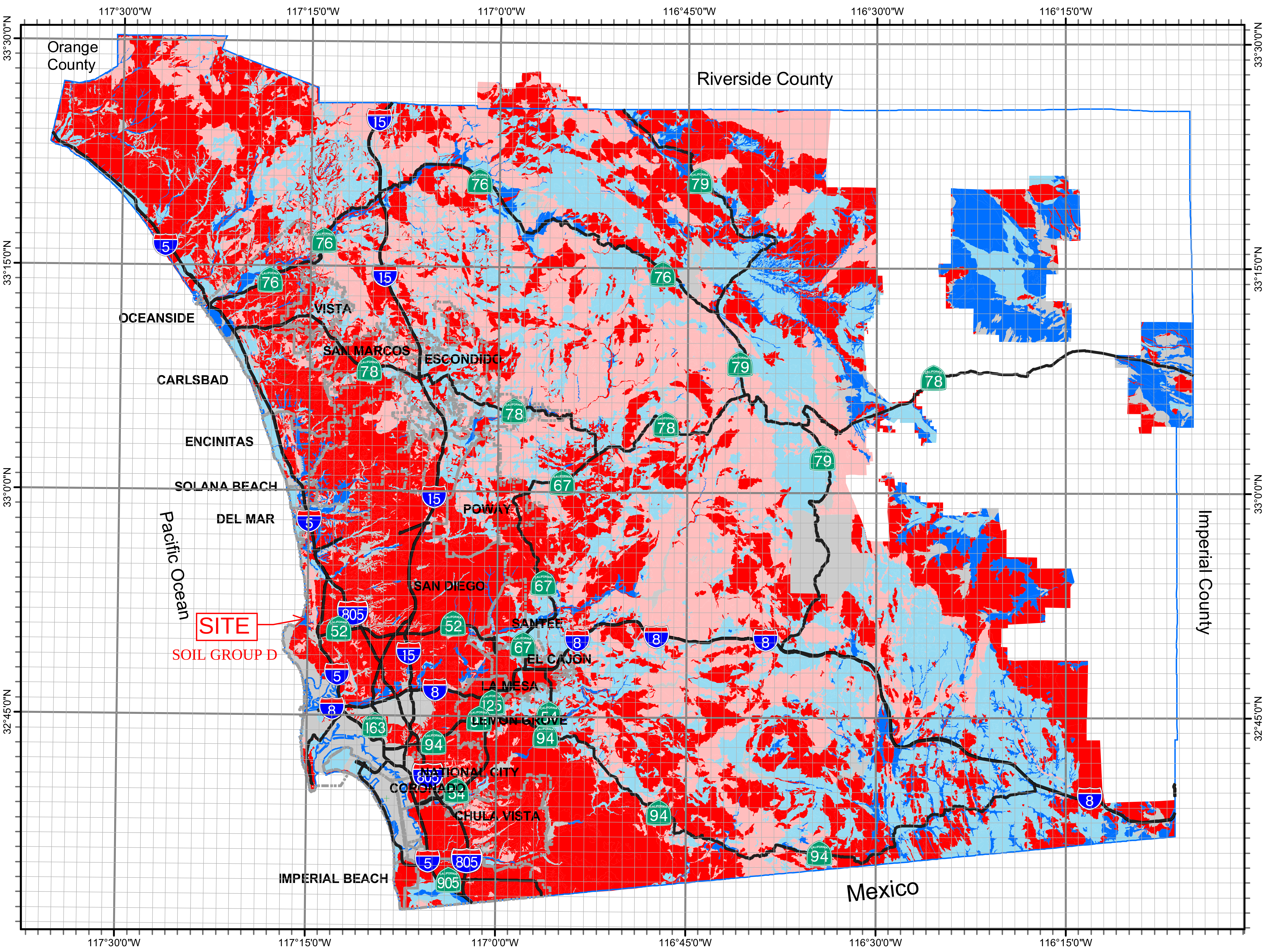
100-yr Required Retention Volume, Basin B-1

Required retention volume $= 4.59*\frac{ft^3}{s}*min*60\frac{s}{min} = 275$ cf

Area between graphs= 4.59

APPENDICES:

County of San Diego Hydrology Manual Soil Hydrologic Group



Legend

- ▬ Major Roads
- ▭ Incorporated City Bdy

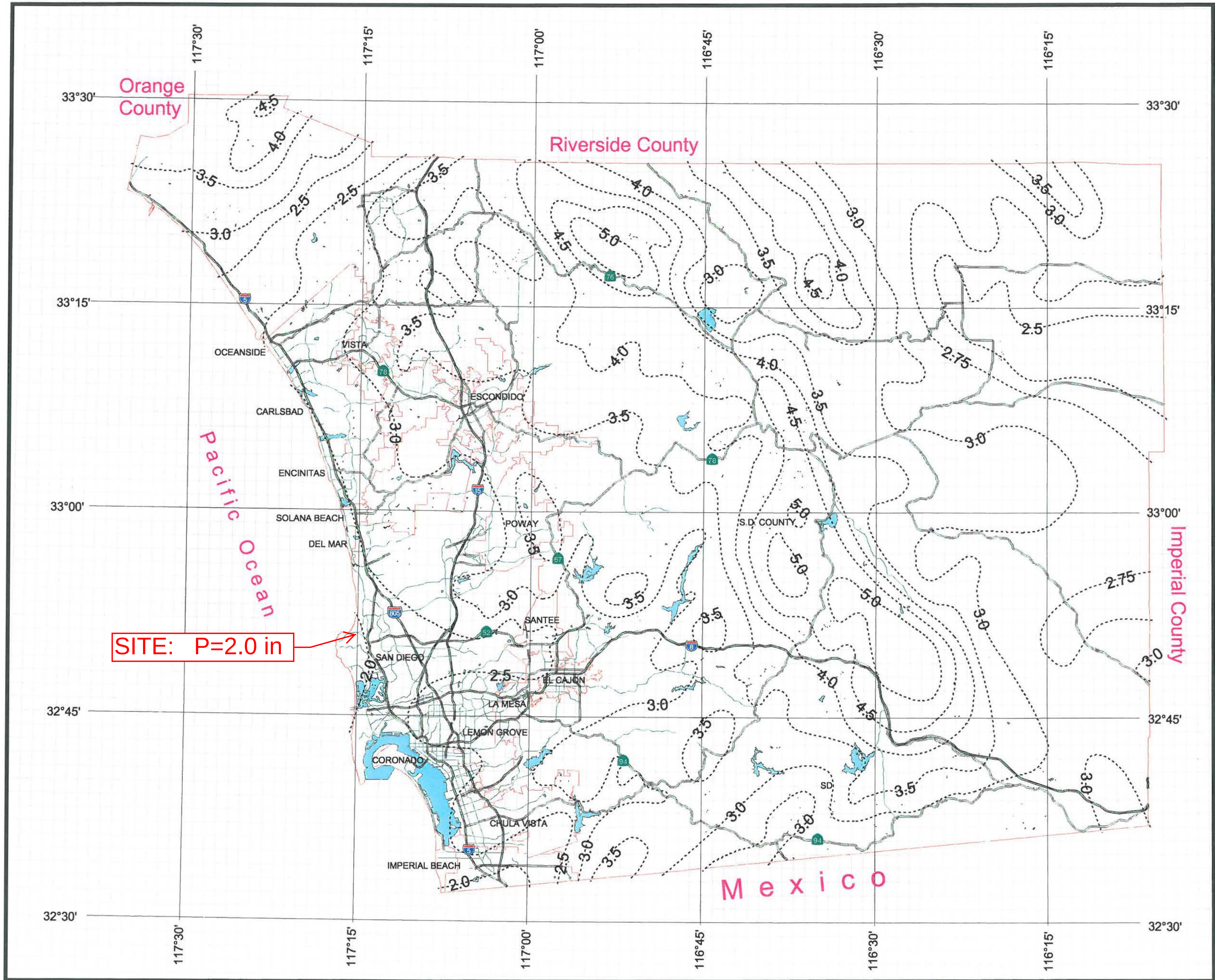
HYDROLOGIC SOIL GROUP

- ⬜ Hydrologic Group Undefined
- 🟦 Hydrologic Group A
- 🟦 Hydrologic Group B
- 🟥 Hydrologic Group C
- 🟥 Hydrologic Group D
- ⬜ No Soil Data

Note: Soil Data Source
USDA/NRCS
SSURGO Soils 2007

N

3 1.5 0 3 Miles



County of San Diego Hydrology Manual

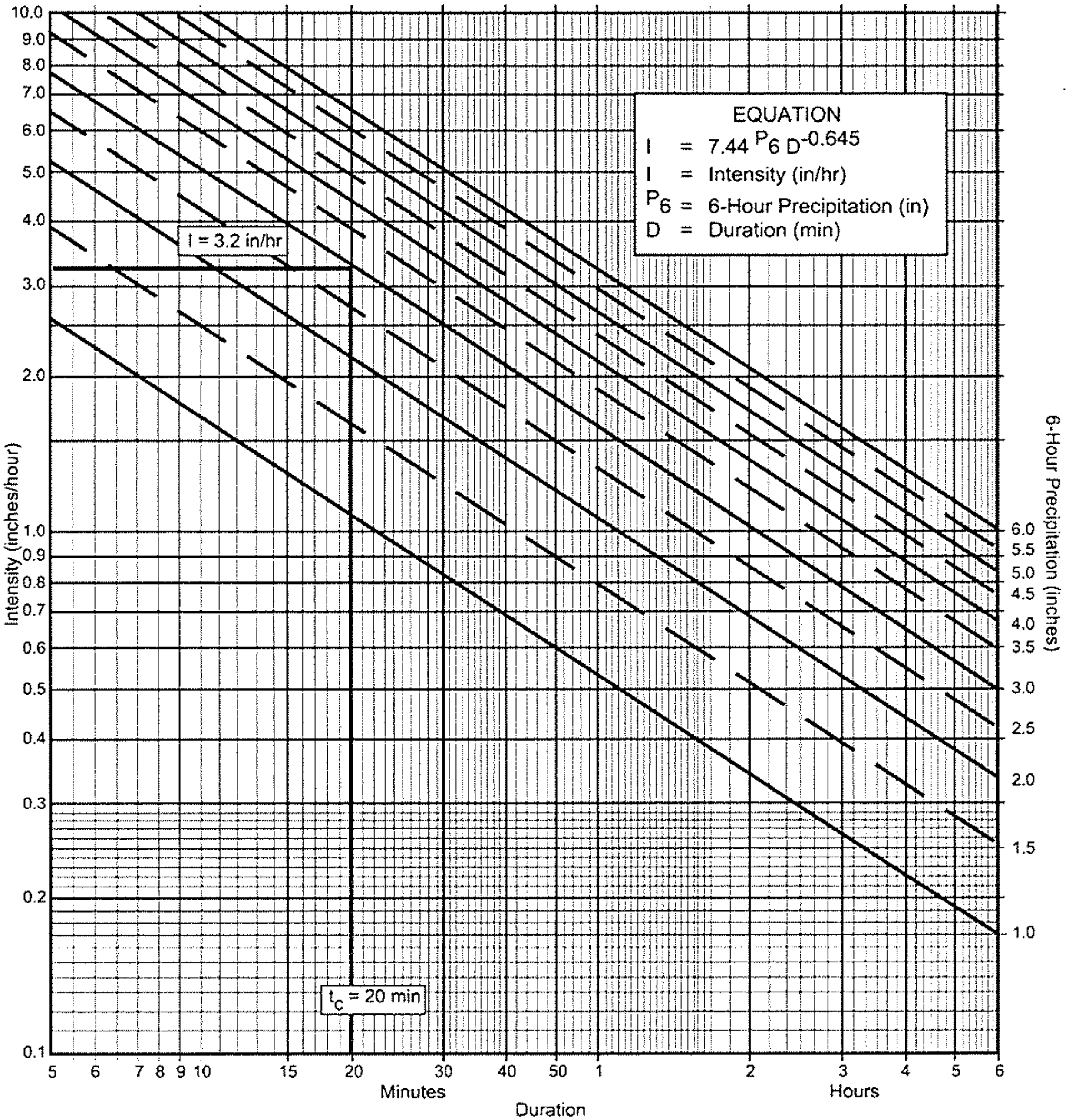
Rainfall Isopluvials

100 Year Rainfall Event - 6 Hours

-------- Isopluvial (inches)

3 0 3 Miles

APPENDIX-4.0



Directions for Application:

(1) From precipitation maps determine 6 hr and 24 hr amounts for the selected frequency. These maps are included in the County Hydrology Manual (10, 50, and 100 yr maps included in the Design and Procedure Manual).

(2) Adjust 6 hr precipitation (if necessary) so that it is within the range of 45% to 65% of the 24 hr precipitation (not applicaple to Desert).

(3) Plot 6 hr precipitation on the right side of the chart.

(4) Draw a line through the point parallel to the plotted lines.

(5) This line is the intensity-duration curve for the location being analyzed.

Application For

(a) Selected frequency ______ year

(b) P_6 = ______ in., P_{24} = ______, $\frac{P_6}{P_{24}}$ = ______ %$^{(2)}$

(c) Adjusted $P_6^{(2)}$ = ______ in.

(d) t_x = ______ min.

(e) I = ______ in./hr.

Note: This chart replaces the Intensity-Duration-Frequency curves used since 1965.

P6 Duration	**1**	**1.5**	**2**	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.5**	**6**
5	2.63	3.95	5.27	6.59	7.90	9.22	10.54	11.86	13.17	14.49	15.81
7	2.12	3.18	4.24	5.30	6.36	7.42	8.48	9.54	10.60	11.66	12.72
10	1.68	2.53	3.37	4.21	5.05	5.90	6.74	7.58	8.42	9.27	10.11
15	1.30	1.95	2.59	3.24	3.89	4.54	5.19	5.84	6.49	7.13	7.78
20	1.08	1.62	2.15	2.69	3.23	3.77	4.31	4.85	5.39	5.93	6.46
25	0.93	1.40	1.87	2.33	2.80	3.27	3.73	4.20	4.67	5.13	5.60
30	0.83	1.24	1.66	2.07	2.49	2.90	3.32	3.73	4.15	4.56	4.98
40	0.69	1.03	1.38	1.72	2.07	2.41	2.76	3.10	3.45	3.79	4.13
50	0.60	0.90	1.19	1.49	1.79	2.09	2.39	2.69	2.98	3.28	3.58
60	0.53	0.80	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2.92	3.18
90	0.41	0.61	0.82	1.02	1.23	1.43	1.63	1.84	2.04	2.25	2.45
120	0.34	0.51	0.68	0.85	1.02	1.19	1.36	1.53	1.70	1.87	2.04
150	0.29	0.44	0.59	0.73	0.88	1.03	1.18	1.32	1.47	1.62	1.76
180	0.26	0.39	0.52	0.65	0.78	0.91	1.04	1.18	1.31	1.44	1.57
240	0.22	0.33	0.43	0.54	0.65	0.76	0.87	0.98	1.08	1.19	1.30
300	0.19	0.28	0.38	0.47	0.56	0.66	0.75	0.85	0.94	1.03	1.13
360	0.17	0.25	0.33	0.42	0.50	0.58	0.67	0.75	0.84	0.92	1.00

Intensity-Duration Design Chart - Example

Table 3-1
RUNOFF COEFFICIENTS FOR URBAN AREAS

Land Use		Runoff Coefficient "C"				
			Soil Type			
NRCS Elements	County Elements	% IMPER.	A	B	C	D
Undisturbed Natural Terrain (Natural)	Permanent Open Space	0*	0.20	0.25	0.30	0.35
Low Density Residential (LDR)	Residential, 1.0 DU/A or less	10	0.27	0.32	0.36	0.41
Low Density Residential (LDR)	Residential, 2.0 DU/A or less	20	0.34	0.38	0.42	0.46
Low Density Residential (LDR)	Residential, 2.9 DU/A or less	25	0.38	0.41	0.45	0.49
Medium Density Residential (MDR)	Residential, 4.3 DU/A or less	30	0.41	0.45	0.48	0.52
Medium Density Residential (MDR)	Residential, 7.3 DU/A or less	40	0.48	0.51	0.54	0.57
Medium Density Residential (MDR)	Residential, 10.9 DU/A or less	45	0.52	0.54	0.57	0.60
Medium Density Residential (MDR)	Residential, 14.5 DU/A or less	50	0.55	0.58	0.60	0.63
High Density Residential (HDR)	Residential, 24.0 DU/A or less	65	0.66	0.67	0.69	0.71
High Density Residential (HDR)	Residential, 43.0 DU/A or less	80	0.76	0.77	0.78	0.79
Commercial/Industrial (N. Com)	Neighborhood Commercial	80	0.76	0.77	0.78	0.79
Commercial/Industrial (G. Com)	General Commercial	85	0.80	0.80	0.81	0.82
Commercial/Industrial (O.P. Com)	Office Professional/Commercial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (Limited I.)	Limited Industrial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (General I.)	General Industrial	95	0.87	0.87	0.87	0.87

*The values associated with 0% impervious may be used for direct calculation of the runoff coefficient as described in Section 3.1.2 (representing the pervious runoff coefficient, Cp, for the soil type), or for areas that will remain undisturbed in perpetuity. Justification must be given that the area will remain natural forever (e.g., the area is located in Cleveland National Forest).

DU/A = dwelling units per acre

NRCS = National Resources Conservation Service

BASIN B-2

FEATURES

Impeller: Cast iron, enclosed, non-clog, dynamically balanced with pump out vanes for mechanical seal protection.

Casing: Cast iron flanged volute type for maximum efficiency. Designed for easy installation on A10-20 slide rail or base elbow rail systems.

Mechanical Seal: SILICON CARBIDE VS. SILICON CARBIDE sealing faces for superior abrasive resistance, stainless steel metal parts, BUNA-N elastomers.

Shaft: Corrosion-resistant, 300 series stainless steel. Threaded design. Locknut on all models to guard against component damage on accidental reverse rotation.

Fasteners: 300 series stainless steel.

Capable of running dry without damage to components.

Designed for continuous operation when fully submerged.

EXTENDED WARRANTY AVAILABLE FOR RESIDENTIAL APPLICATIONS.

WS_BHF Series

Model 3887BHF

SUBMERSIBLE SEWAGE PUMP

GOULDS
WATER TECHNOLOGY
a xylem brand

APPLICATIONS

Specifically designed for the following uses:

- Homes
- Sewage systems
- Dewatering/Effluent
- Water transfer
- Light industrial
- Commercial applications

Anywhere waste or drainage must be disposed of quickly, quietly and efficiently.

SPECIFICATIONS

Pump

- Solids handling capabilities: 2" maximum
- Capacities: up to 220 GPM
- Total heads: up to 81 feet TDH
- Discharge size: 2" NPT threaded companion flange as standard. 3" option available but must be ordered separately. (Order no. A1-3)
- Temperature: 104°F (40°C) continuous
  140°F (60°C) intermittent.

MOTORS

- Fully submerged in high grade turbine oil for lubrication and efficient heat transfer. All ratings are within the working limits of the motor.

Class B insulation on ⅓-1½ HP models.

Class F insulation on 2 HP models.

Single phase (60 Hz):

- Capacitor start motors for maximum starting torque.
- Built-in overload with automatic reset.
- SJTOW or STOW severe duty oil and water resistant power cords.
- ⅓ – 1 HP models have NEMA three prong grounding plugs.
- 1½ HP and larger units have bare lead cord ends.

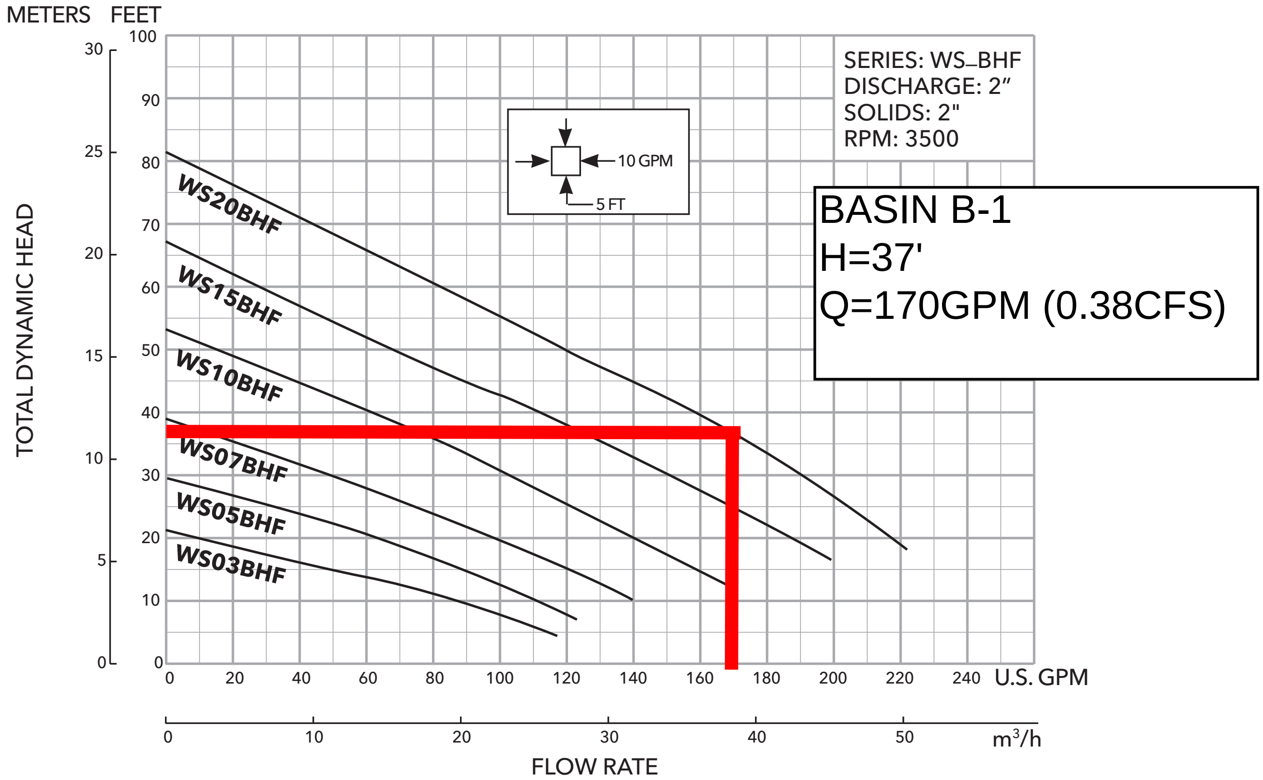
Three phase (60 Hz):

- Class 10 overload protection must be provided in separately ordered starter unit.
- STOW power cords all have bare lead cord ends.

- Bearings: Upper and lower heavy duty ball bearing construction.
- Designed for Continuous Operation: Pump ratings are within the motor manufacturer's recommended working limits, can be operated continuously without damage when fully submerged.
- Power Cable: Severe duty rated, oil and water resistant. Epoxy seal on motor end provides secondary moisture barrier in case of outer jacket damage and to prevent oil wicking. Standard cord is 20'. Optional lengths are available.
- Motor Cover O-ring: Assures positive sealing against contaminants and oil leakage.

AGENCY LISTINGS

Tested to UL 778 and CSA 22.2 108 Standards
By Canadian Standards Association
File #LR38549



BASIN B-1
H=37'
Q=170GPM (0.38CFS)

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ATTACHMENT 6
GEOTECHNICAL AND GROUNDWATER INVESTIGATION REPORT

Attach project's geotechnical and groundwater investigation report. Refer to Appendix C.4 to determine the reporting requirements.

Geotechnical Exploration, Inc.

SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

04 August 2016

Trevor and Staci Klein
2585 Calle del Oro
La Jolla, CA 92037

Job No. 13-10407

Subject: **Addendum Geotechnical Report Response to City Reviewer**
Klein Residence
2585 Calle del Oro
La Jolla, California

Dear Mr. and Mrs. Klein:

In accordance with your request and as required by LDR-Geology Reviewer, we are replying to comments in a memo with a completion date of July 28, 2016 (Cycle 6). The LDR reviewer has reviewed our Limited Geotechnical Investigation report dated February 10, 2014 and our Addendum Geotechnical Report Response to City Reviewer, dated June 28 2016, as well as a Site Development Plan by Studio William Hefner Architecture dated July 1, 2016.

Issue No. 10: Storm Water Requirements for proposed conceptual development will be evaluated by LDR-Engineering review. Priority Development Projects (PDPs) may require an investigation of storm water infiltration feasibility in accordance with the Storm Water Standards (including Appendix C and D). Check with your LDR-Engineering reviewer on requirements. LDR-Engineering may determine that LDR-Geology review of a storm water infiltration evaluation is required. (New Issue).

GEI Response: We are in the process of performing an investigation of storm water infiltration feasibility and although infiltration may be feasible, it is our opinion that any long term infiltration may result in geotechnical hazards which cannot be reasonably mitigated to an acceptable level.

Issue No. 11: The geotechnical consultant indicates that an addendum report providing the logs of the subsurface exploration that provides the detailed direct observation and mapping of the bedding attitudes conducted by an engineering geologist is in progress. Submit a copy of that report for review. (New Issue).

GEI Response: In our geotechnical report dated February 10, 2014, we provided boring logs of subsurface exploration at the site. The borings were performed with small diameter augers and obtained 3-inch-diameter soil samples. No large diameter borings were excavated since they were not considered necessary for the current remodel. We observed nearby bedrock exposures and reviewed the geological map by Kennedy and Tan (2008) that indicated bedding attitudes in this area were not unfavorable, with strikes generally N30°W to N65°W and dips northeast at angles of 3 to 5 degrees, with direction parallel to the hillside and not out of slope. We confirmed the reported bedding attitudes in our supplemental test pits placed in the eastern portion of the site on July 18, 2016. We encountered the Ardath Shale Formation at a depth of 1 to 2 feet and measured strikes generally N60W and dips 3 degrees northeast. The bedding dips into the hillside and is considered to be favorable (refer to Appendix A, Slope Stability Analysis).

Issue No. 12: *Submit original quality prints and digital copies (on CD/DVD/or USB data storage device) of the referenced and requested geotechnical reports for our records. (New Issue).*

GEI Response: We are providing a quality copy of this report and a copy on CD.

If you have further questions regarding this letter, please contact our office. Reference to our **Job No. 13-10407** will help expedite a response to your inquiry.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

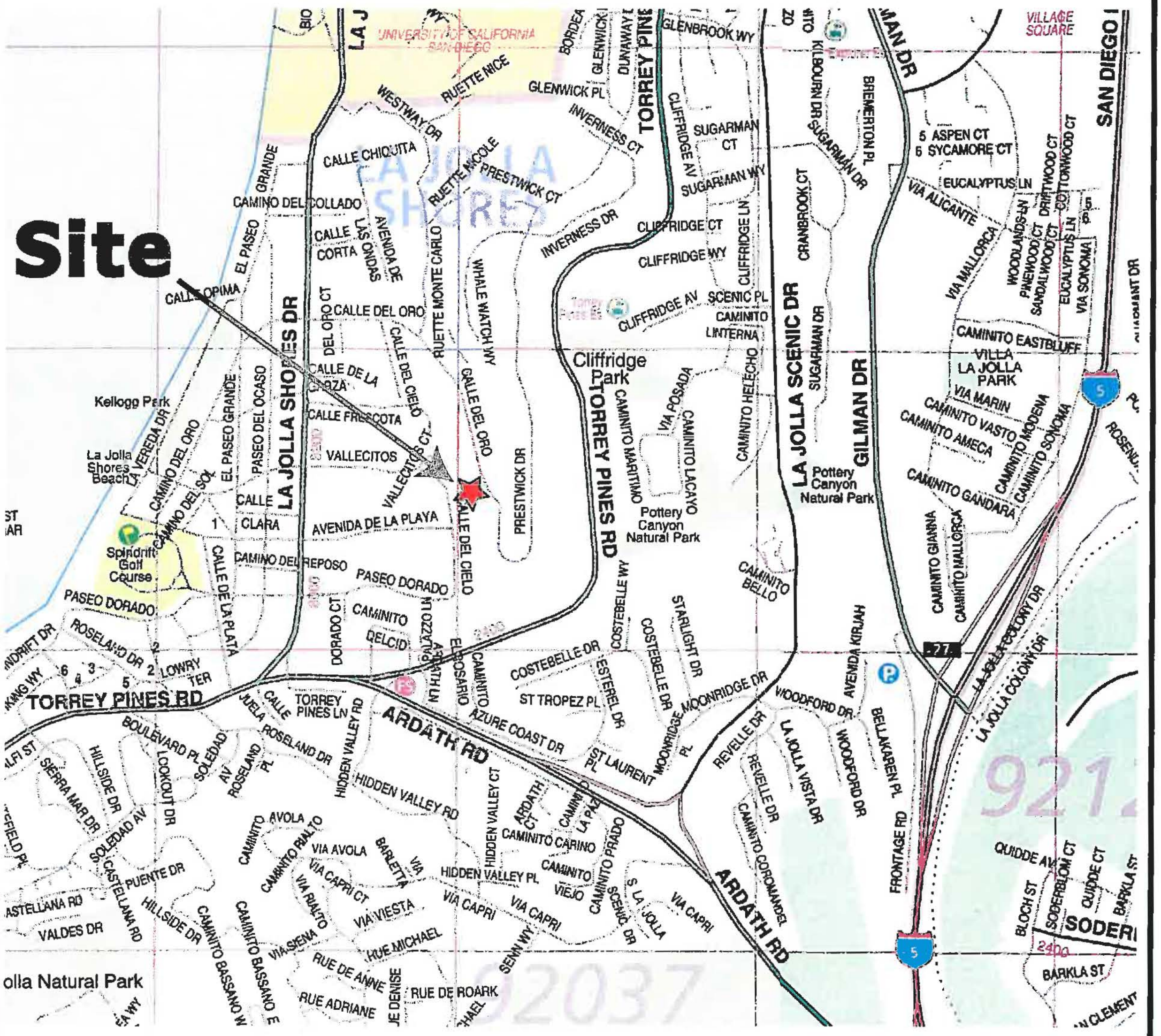
Jaime A. Cerros, P.E.
R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

Jonathan A. Browning
C.E.G. 2615/P.G. 9012
Senior Project Geologist

JAIME A. CERROS
No. 002007
Exp 9/30/17

JONATHAN A. BROWNING
No. 2615

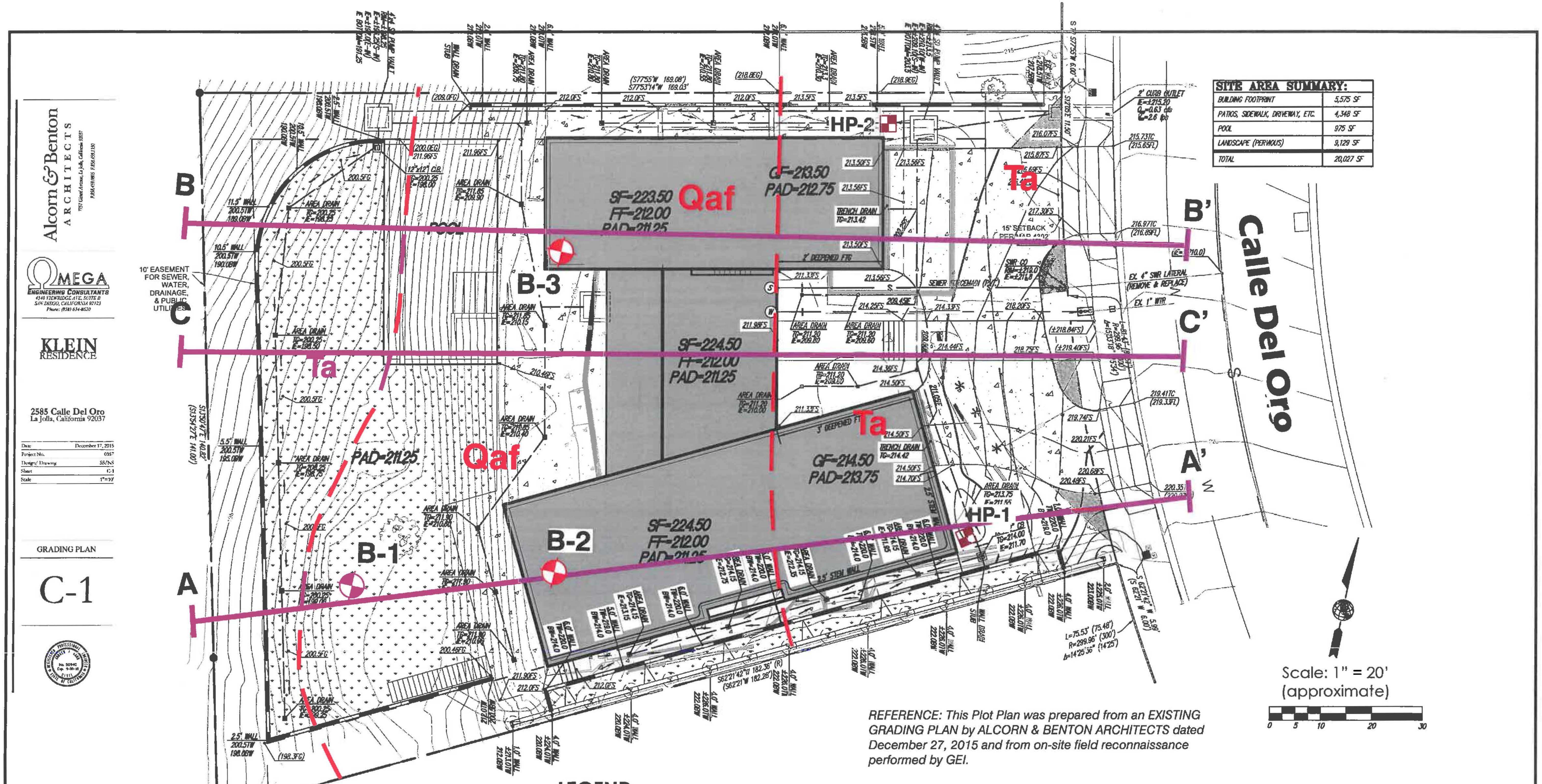
VICINITY MAP



Thomas Bros Guide San Diego County pg 1227-H6

Proposed Klein Residence
2585 Calle Del Oro
La Jolla, CA.

Figure No. I
Job No. 13-10407



REFERENCE: This Plot Plan was prepared from an EXISTING GRADING PLAN by ALCORN & BENTON ARCHITECTS dated December 27, 2015 and from on-site field reconnaissance performed by GEI.

GEOLOGIC LEGEND

Qaf	Artificial Fill
Ta	Ardath Shale Formation
— —	Approximate Geologic Contact

LEGEND

⊕ B-3	Approximate Location of Exploratory Boring
▨ HP-3	Approximate Location of Exploratory Handpit
A ├──┤ A'	Approximate Line of Cross Section

PLOT PLAN

Proposed Klein Residence
2585 Calle Del Oro
La Jolla, CA.
Figure No. II
Job No. 13-10407

Geotechnical Exploration, Inc.

July 2016

13-10407-P3.ai

EQUIPMENT **Hand Tools**	DIMENSION & TYPE OF EXCAVATION **3' X 3' X 3' Handpit**	DATE LOGGED **7-18-16**
SURFACE ELEVATION **± 220' Mean Sea Level**	GROUNDWATER/ SEEPAGE DEPTH **Not Encountered**	LOGGED BY **JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
1			**CLAYEY SAND**, with abundant roots and rock fragments. Loose to medium dense. Damp. Gray-brown. **FILL (Qaf)**	SC								
2			**SANDY CLAY**, with some gypsum crystals. Stiff to hard. Damp. Light gray and yellow-brown. **ARDATH SHALE FORMATION (Ta)**	CL								
3 4			Bottom @ 3'									

▼ PERCHED WATER TABLE

☒ BULK BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S NUCLEAR FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME **Klein Residence**		
SITE LOCATION **2585 Calle del Oro, La Jolla, CA**		
JOB NUMBER **13-10407**	REVIEWED BY **LDR/JAC**	LOG No. **HP-1**
FIGURE NUMBER **IIId**	Geotechnical Exploration, Inc.	

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Hand Tools	3' X 3' X 3' Handpit	7-18-16
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 220' Mean Sea Level	Not Encountered	JKH

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–1			**CLAYEY SAND**, with some roots and rock fragments. Loose to medium dense. Damp. Gray-brown. **FILL (Qaf)**	SC									
1			**SANDY CLAY**, with some gypsum crystals; moderately fractured. Very stiff to hard. Damp. Yellow-brown. **ARDATH SHALE FORMATION (Ta)**	CL									
2		☒	-- 85% passing #200 sieve.				15.5	112.6			48		
			Bedding: N60°W, 3°NE.										
3			Bottom @ 3'										
4													

▼	PERCHED WATER TABLE
☒	BULK BAG SAMPLE
1	IN-PLACE SAMPLE
■	MODIFIED CALIFORNIA SAMPLE
S	NUCLEAR FIELD DENSITY TEST
▨	STANDARD PENETRATION TEST

JOB NAME
Klein Residence

SITE LOCATION
2585 Calle del Oro, La Jolla, CA

JOB NUMBER
13-10407

REVIEWED BY
LDR/JAC

Geotechnical Exploration, Inc.

FIGURE NUMBER
IIIe

LOG No.
HP-2