soil PACIFIC INC.
Geotechnical and Environmental Services

January 22, 2018
Project No. A- 6595-17

West Mesa Investment Group LLC
425 E. 5th Street
Long Beach, California

Subject: **Soil and Foundation Evaluation Report**
Proposed Multi- Family Residential Building
425 E. 5th Street, Long Beach, California

Dear Sir:

Pursuant to your authorization, we are pleased to submit our report for the subject project. Our evaluation was conducted in January 2018. This evaluation consists of field exploration; sub-surface soil sampling; laboratory testing; engineering evaluation and preparation of the following report containing a summary of our conclusions and recommendations.

The opportunity to be of service is appreciated. Should any questions arise pertaining to any portion of this report, please contact this firm in writing for further clarification.

Respectfully submitted,

Soil Pacific, Inc.

Yones Kabir
President

Hoss Eftekhari
RCE

REGISTERED PROFESSIONAL ENGINEER
HOSS EFTEKHARI
No. C50121
EXP. 6-30-19
CIVIL
STATE OF CALIFORNIA

Soil and Foundation Evaluation Report
Proposed Multi- Family Residential Building
425 E. 5th Street, Long Beach, California

Prepared For:

West Mesa Investment Group LLC
425 E. 5th Street
Long Beach, California

Prepared by:

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January 22, 2018
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Soil and Foundation Evaluation Report
Proposed Multi- Family Residential Building
425 E. 5th Street, Long Beach, California

LIMITATIONS

Between exploratory excavations and/or field testing locations, all subsurface deposits, consequent of their anisotropic and heterogeneous characteristics, can and will vary in many important geotechnical properties. The results presented herein are based on the information in part furnished by others and as generated by this firm, and represent our best interpretation of that data benefiting from a combination of our earthwork related construction experience, as well as our overall geotechnical knowledge. Hence, the conclusions and recommendations expressed herein are our professional opinions about pertinent project geotechnical parameters which influence the understood site use; therefore, no other warranty is offered or implied.

All the findings are subject to field modification as more subsurface exposures become available for evaluations. Before providing bids, contractors shall make thorough explorations and findings. Soil Pacific Inc., is not responsible for any financial gains or losses accrued by persons/firms or third party from this project.

In the event the contents of this report are not clearly understood, due in part to the usage of technical terms or wording, please contact the undersigned in writing for clarification.

SECTION 1.0
PRELIMINARY EVALUATION

1.1 Site Description

The subject property is in the East Village portion of the community of Long Beach , California. The site is set at 425 E. 5th Street, Long Beach, California. The lot is rectangular with the elongated axis in a north-south direction.

The lot has been previously graded and developed, as a multi -family residence. The main building is located in the south part of the lot. The property is limited at the north by E. Cereza Way and the south by 5th Street,. The eastern property is a vacant land currently used as a paved parking lot and the western side is limited by N. Frontenac Court. Site elevation is about 35 above MSL. Site sheet flow is toward the south.

1.2 Planned Land Use

It is understood that the proposed development will consist of site renovation and building a newly designed building structure. Existing buildings/garage will be demolished upon securing the permit for new building.

1.3 Field Exploration

Subsurface conditions were explored by excavation of two auger boring to a minimum depth of 20 feet below the existing grade. Native materials discovered at shallow depth . Thickness of uncertified soils such as residual soil /top soils is limited to 2 feet where the borings were drilled.

Earth materials encountered within the exploratory borings were classified and logged by firm staff engineer in accordance with the visual-manual procedures of the Unified Soil Classification System (USCS), ASTM Test Standard D2488. Following our exploration, borings were loosely backfilled with the soil cuttings. The approximate locations of the exploratory borings are shown on the Exploration Location Map Figure A-1-1. Descriptive boring logs are presented in Appendix A.

1.4 Laboratory Testing

1.4.1. Classification

Soils were classified visually according to the Unified Soil Classification System. Moisture content and dry density determinations have been made for the samples taken at various depths in the exploratory excavations. Results of moisture-density and dry-density determinations, together with classifications, are shown on the Test-pit logs, Appendix A.

1.4.2 Expansion

An expansion index test was performed on a representative sample in accordance with the California Building Code Standard . No expansion potential (EI=12) is anticipated for the encountered native soils at the proposed sub-grade elevation.

1.4.3 Direct Shear

Shear strength parameters are determined by means of strain-controlled, double plain, direct shear tests performed in general accordance with ASTM D-3080. Generally, three or more specimens are tested, each under a different normal load, to determine the effects upon shear resistance and displacement, and strength properties such as Mohr strength envelopes . The direct shear test is suited to the relatively rapid determination of consolidated drained strength properties because the drainage paths through the test specimen are short, thereby allowing excess pore pressure to be dissipated more rapidly than with other drained stress tests. The rate of deformation is determined from the time required for the specimen to achieve fifty percent consolidation at a given normal stress. The test can be made on all soil materials and undisturbed, remolded or compacted materials. There is however, a limitation on maximum particle size. Sample displacement during testing may range from 10 to 20 percent of the specimen's original diameter or length.

The sample's initial void ratio, water content, dry unit weight, degree of saturation based on the specific gravity, and mass of the total specimen may also be computed. The shear test results are plotted on the attached shear test diagrams and unless otherwise noted on the shear test diagram, all tests are performed on undisturbed, saturated samples.

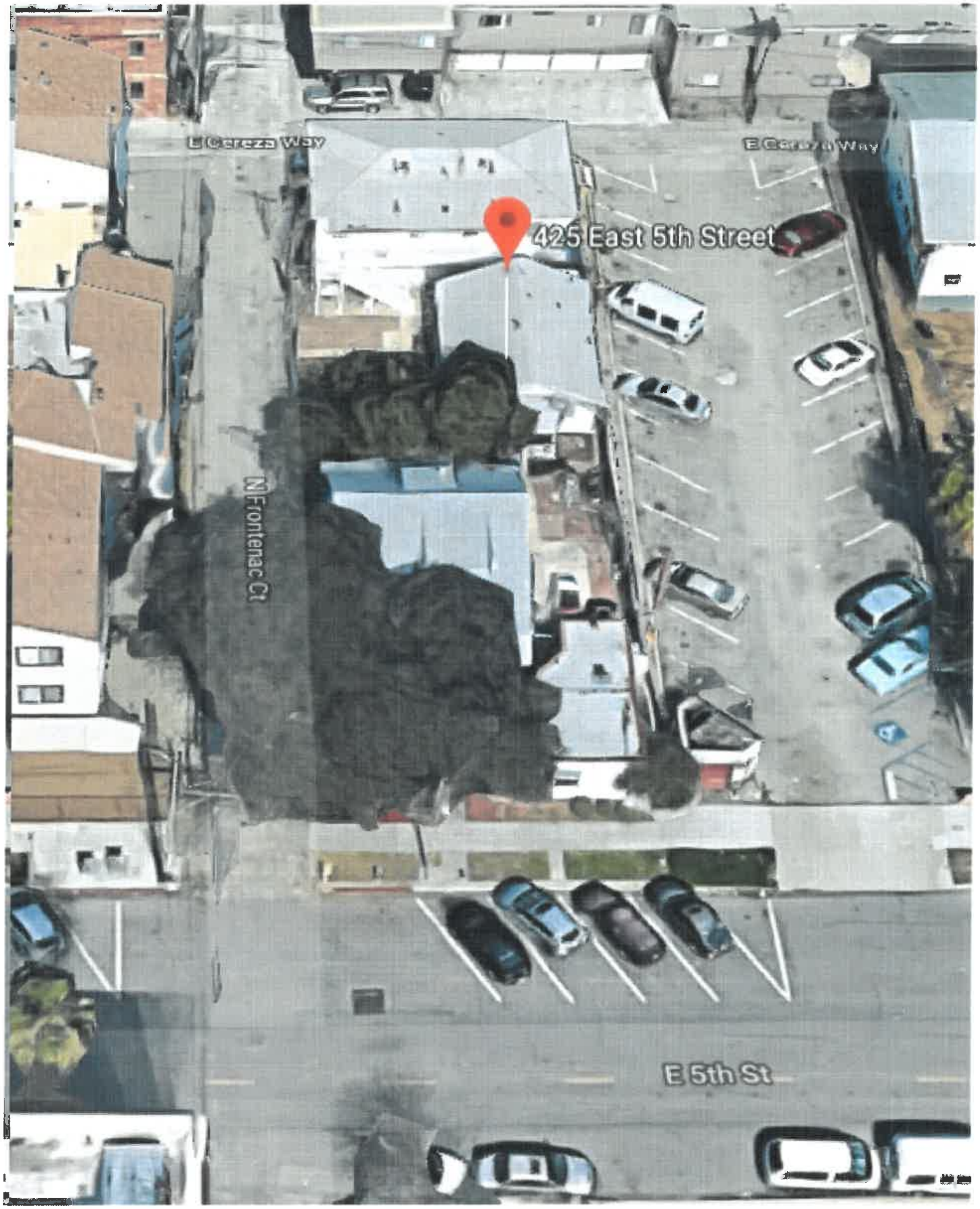

Figure 1: Site aerial photo.

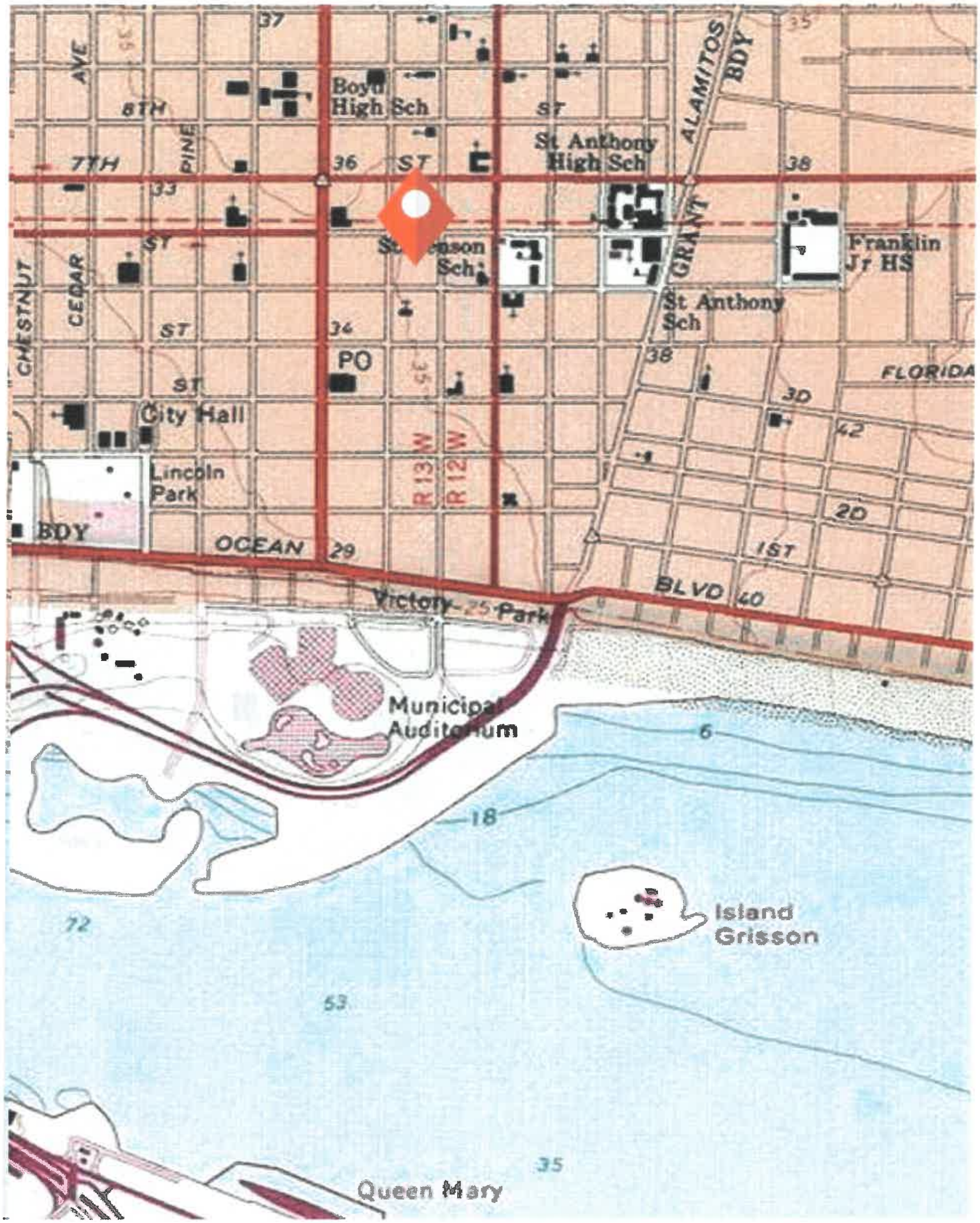

Figure 2: Site Topographic Map. USGS /AAGS

Section 2.0
Conclusions

The proposed construction is considered feasible from a soils engineering standpoint. All earthwork should be performed in accordance with applicable engineering recommendations presented herein or applicable Agency Codes, whichever are the most stringent.

2. 1 Earth Materials

The site is mostly underlain by light brown to olive fine grained silty sand and sandy soils. Encountered soils were mainly damp to moist and firm in place.

The depth of topsoil/fill mantel may varies throughout the site, however the fill mantel at the site was limited to 2 feet where boring logs were performed. Maximum depth of explored boring at the site is 20 feet.

2.2 Foundations

All foundation will be embedded into the same type of engineered fill soils. All newly designed isolated pad or continuous foundation must be embedded into firm and approved engineered soils. Cut and fill transition is not allowed.

2.3 Bearing Materials

The surficial soils up to 2 feet are disturbed and inadequate from a soil engineering standpoint.

2.4 Groundwater

The site is located within a mile north of Coastal Plain of Los Angeles-Santa Monica, (California Department of Water Resources, [CDWR], 2016). Groundwater depth varies within the area and flow direction beneath the subject site is toward the south-southwest. No groundwater wells were listed on the property; however, several groundwater wells are listed in the site vicinity.

During our investigation, free groundwater was not encountered within the limited 20 feet of sub-surface exploration below the existing grade.. The depth of groundwater may fluctuate depending upon the time and period of the year and estimated to be over 40 feet depth.

2.5 CBC Seismic Design Parameters

Earthquake loads on earthen structures and buildings are a function of ground acceleration, which may be determined from the site-specific acceleration response spectrum. To provide the design team with the parameters necessary to construct the site-specific acceleration response spectrum for this project, we used two computer applications that are available on the United States Geological Survey (USGS) website, http://geohazards.usgs.gov/.

Specifically, the Design Maps website http://geohazards.usgs.gov/designmaps/us/application.php was used to calculate the ground motion parameters. And, the 2008 PSHA Interactive Deaggregation website http://geohazards.usgs.gov/deaggint/2008/ was used to determine the appropriate earthquake magnitude.

The printout attached in Appendix C provides parameters required to construct the site-specific acceleration response spectrum based on the 2016 CBC guidelines.

2.6 Chemical Contents

Chemical testing for detection of hydrocarbon or other potential contamination is beyond the scope of this report.

2.7 Liquefaction Study/ Secondary Seismic Hazard Zonation

Based on our review of the published 7.5-minute quadrangle Hazard maps, the subject site is not located within an area having a potential for Liquefaction susceptibility. Liquefaction occurs when seismically-induced dynamic loading of a saturated sand or silt causes pore water pressures to increase to levels where grain-to-grain contact pressure is significantly decreased and the soil material temporarily behaves as a viscous fluid. Liquefaction can cause settlement of the ground surface, settlement and tilting of engineered structures, flotation of buoyant buried structures and fissuring of the ground surface. A common manifestation of liquefaction is the formation of sand boils (short-lived fountains of soil and water emerges from fissures or vents and leave freshly deposited conical mounds of sand or silt on the ground surface). Lateral spreading can also occur when liquefaction occurs adjacent to a free face such as a slope or stream embankment.

The types of seismically induced flooding that may be considered as potential hazards to a particular site normally includes flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major reservoir or other water retention structure upstream of the site. Since the site has an average elevation of approximately 35 feet above sea level, and since it does not lie in close proximity to an enclosed body of water, the probability of flooding from a tsunami or seiche is considered to be low. In addition, the site is not located within a designated tsunami inundation area

2.8 Foundations

Proposed building will be 5story building therefore special such as mat foundation embedded into approved and engineered fill soils formation will be used to support the proposed structure.

2.9 Bearing Materials

Based on our findings, the existing top soils are not considered an adequate , therefore, only approved engineered fill soils will be considered as bearing materials. Cut and fill transition is not allowed.

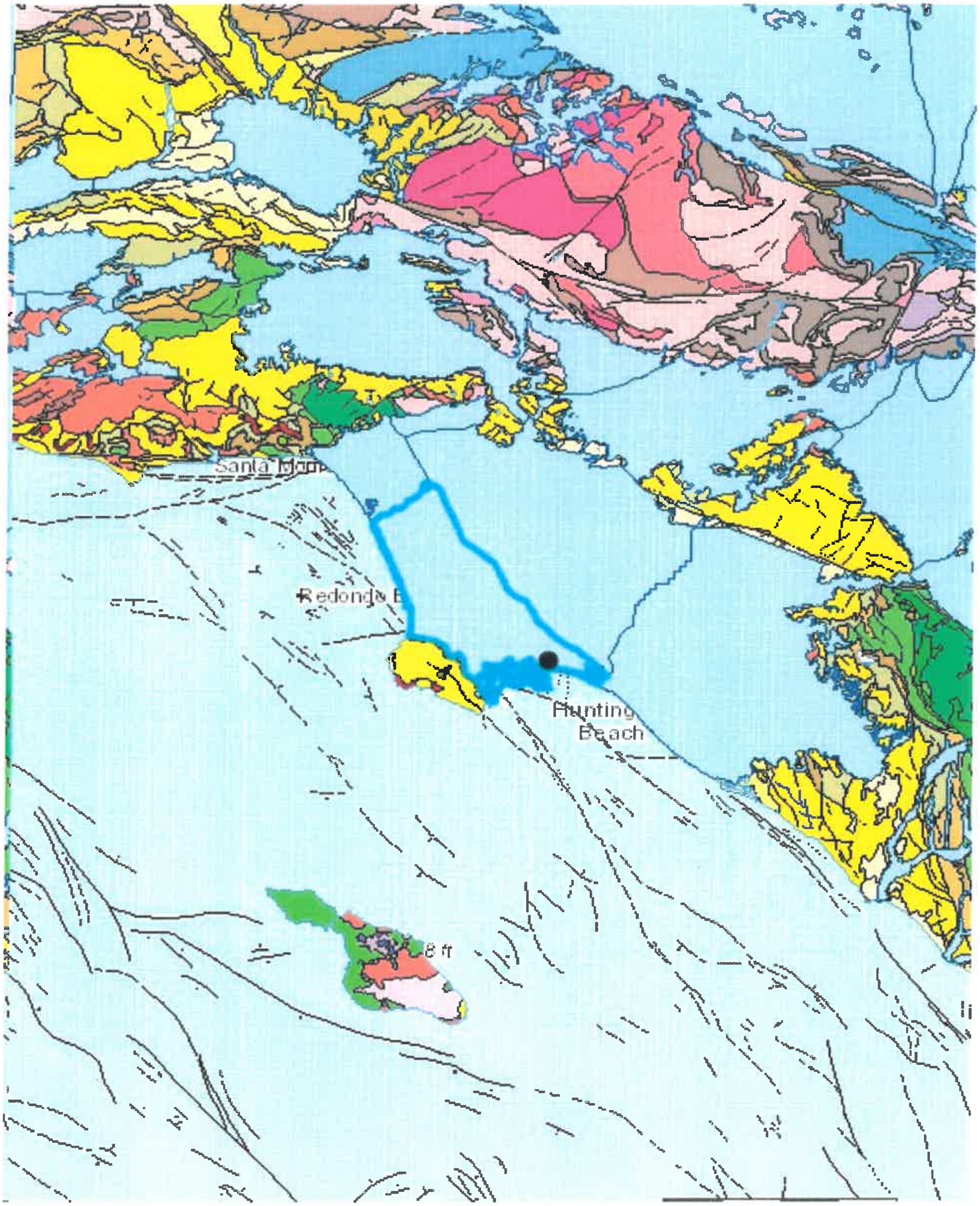

Figure 3: Site Groundwater Boundary Map.

Section 3.0
Recommendations

Based on our site exploration and experience with similar projects, the proposed construction is considered feasible from a soils engineering standpoint provided the following recommendations are made part of the plans and should be implemented during construction.

3.1 Clearing and Site Preparation

Grading plan for the site or full architectural plan is not available for review, therefore as a courtesy we provided a preliminary recommendations. These recommendations may be used for site grading plan preparation if intended to use. During demolishing of the existing building/s if any unanticipated subsurface improvements (pipe lines, irrigation lines, etc.) are encountered then this office should be informed and appropriate remedial recommendations would subsequently be provided.

The excavated onsite materials are considered satisfactory for reuse in the fill if the moisture content is near the optimum. All organic material and construction debris should be removed and shall be segregated. Any imported fill should be observed, tested, and approved by the soils engineer prior to use as fill. Rocks larger than 6 inches in diameter should not be used in the fill.

1. The areas to receive compacted fill should be stripped of all vegetation, construction debris and trashes, non engineered fill, left in place incompetent material up to approved soils. If soft spots are encountered, a project soil engineer will evaluate the site conditions and will provide necessary recommendations.

2. The exposed grade should then be overexcavated to a minimum of 3.5 feet. The excavated area should be scarified to a minimum of 8 inches, adjusted to optimum moisture content, and reworked to achieve a minimum of 90 percent relative compaction. Overexcavation within 5 feet of the adjacent buildings or public way require shoring or slot cut method A, B, and C.

3. Compacted fill should extend at least 5 feet beyond all perimeter footings or to a distance equal to the depth of the certified compacted fill, whichever is the greatest and feasible.

4. Compacted fill, consisting of on-site soil shall be placed in lifts not exceeding 6 inches in uncompacted thickness. The excavated onsite materials are considered satisfactory for reuse in the fill if the moisture content is near optimum. All organic material and construction debris should be removed and shall be segregated. Any imported fill should be observed, tested, and approved by the soils engineer prior to use as fill. Rocks larger than 6 inches in diameter should not be used in the fill.

5. The fill should be compacted to at least 90 percent of the maximum dry density for the material. The maximum density should be determined by ASTM Test Designation D 1557-00.

6. Field observation and compaction testing during the grading should be performed by a representative of Soil Pacific Inc. to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compaction effort should be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent relative compaction is obtained.

3.2 Stability of Temporary Cuts

The stability of temporary cuts required during the removal process depends on many factors, including the slope angle, the shearing strength of the underlying materials, the height of the cut, and the length of time the excavation remains open and exposed to equipment vibrations and rainfall. The geotechnical consultant should be present to observe all temporary excavations at the site. The possibility of temporary excavations failing may be minimized by:

1) keeping the time between cutting and filling operations to a minimum;
2) limiting excavation length exposed at any one time; and,
3) cutting no steeper than a 1: 1 (h:v) inclination for cuts in excess of 4 feet in height.
4) or shoring prior to cut.

Any excavation having a horizontal distance equal to depth of the excavation with an adjacent structural building will not require shoring. Otherwise, the excavation must be shored. However, in accordance with CALOSHA, the maximum height of uncharged excavation will not exceed 5 feet. Any excavation exceeding 3 feet vertical will be trimmed to a minimum of 1:1 slope.

3.3 Foundations

The following recommendations may be used in preparation of the design and construction of the foundation system.

3.3.1 Bearing Value

As specified above, the proposed building structure shall be constructed on Mat foundation placed on the firm engineered soils.

The allowable bearing value for conventional footings, having a minimum diameter of 24 inches and a minimum embedment of 2 feet into approved soils , should not exceed 2500 pounds per square foot. This value may be increased by one-third for short duration (wind or seismic) loading.

3.3.2 Mat Foundation

The contact pressure at the bottom of mat foundation of the building having a 2-foot thick, uniformly loaded concrete mat will be designed. This includes the weight of the mat itself. An allowable bearing capacity of 2,500 pounds per square foot bearing value of compacted soil should not be exceeded. This bearing value is for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces.

Since the recommended bearing value is a net value, the weight of concrete in the mat may be taken as 150 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the mat.

For design purposes, it is recommended that a modulus of subgrade reaction of 150 kips per cubic foot (kcf) be utilized for the subgrade soils.

All mat footing excavations should be observed by personnel of our firm to verify penetration into recommended bearing materials. Footings should be deepened, if necessary, to extend into satisfactory soils. Footing excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required footing backfill should be mechanically compacted; flooding is not permitted.

3.3.3 Foundation Settlement

Based upon anticipated structural loads, the maximum total settlement for the proposed foundation is not expected to exceed 1 inch. Differential settlement between adjacent footings and lateral displacement of lateral resisting elements should not exceed ½ inch.

3.3.4Concrete Type

Based on experience with similar projects in the area, concrete Type II may be used. However, upon completion of planned footing excavations, representative samples of exposed materials may be tested for soluble sulfate in order to confirm the recommended Type II concrete mix.

3.4 Utility Trenches Backfill

Utility trenches backfill should be placed in accordance with Appendix D. It is the owners' and contractors' responsibility to inform subcontractors of these requirements and to notify Soil Pacific when backfill placement is to begin.

3.5 Seismic Design and Construction

Construction should be in conformance with seismic design parameters of the latest edition of California Building Code. Please refer to Appendix C for seismic design recommendation in accordance with CBC code recommendations.

3.6 Surface and Sub-surface Drainage Provisions

Proper surface drainage gradients are helpful in conveying water away from foundations and other improvements. Subsurface drainage provisions are considered essential in order to reduce pore-pressure build-up behind retaining structures. Ponding of water enhances infiltration of water into the local soils and should not be allowed anywhere on the pad.

3.7 Drainage Control

Patio subgrade soil should be compacted to a minimum of 90 percent to a depth of 18 inches. All run-off should be gathered in gutters and conducted off site in a non-erosive manner. Planters located adjacent to footings should be sealed and leach water intercepted.

3.8 Slabs-on-grade

Patio Slabs should be a minimum of 5 inches thick. Slab areas that are to be carpeted or tiled, or where the intrusion of moisture is objectionable, should be underlain by a moisture barrier consisting of 10-mil Visqueen, properly protected from puncture by four inches of crushed rock per Calgreen requirement.

3.9 Reinforcement

Observation and classification of soil samples recovered from the site indicate the potential for expansion is null at surficial soils. Based on this, footings should include a minimum of two No.5 steel bars, placed at the top and two No.5 bars at the bottom, and slabs should have No. 3 rebars properly located at the center of the thickness.

3.10 Conventional Retaining Wall

If a conventional retaining wall is planned, the following design criteria may be used:

1) Where a free standing structure is proposed, a minimum equivalent fluid pressure, for lateral soil loads, of 41 pounds per cubic foot, may be used as design for onsite non-expansive granular soils conditions and level backfill (10:1 or less). If the wall is restrained against free movement (= ±1 % of wall height) then the wall should be designed for lateral soil loads approaching the at-rest condition. Thus, for restrained conditions, the above value should be increased to 62 pcf. In addition, all retaining structures should include the appropriate allowances for any anticipated surcharge loads.

2) An allowable soil bearing pressure of 2500 lbs. per square foot may be used in design for footings embedded to a minimum of 24 inches below the lowest adjacent competent grade and embedded into bedrock.

3) A friction coefficient of 0.4 between concrete and natural or compacted soil and a passive bearing value 345 lbs. per square foot per foot of depth, up to a maximum of 4,000 pounds per square foot at the bottom excavation level may be employed to resist lateral loads.

Back drain system will consist of a free-draining material made up of at least 1 cubic foot of 3/4-inch crushed rock/gravel around pipe drains. If an open space greater than 1 foot exists between the back of the wall and the soil face, gravel backfill should be compacted by vibration. An impervious soil cap should be provided at the top of the wall backfill to prevent infiltration of surface water into the back drain system. The cap may be a combination of concrete and/or compacted fine grained soils. The compacted backfill soil cap should be at least 1 foot thick when used in conjunction with a concrete slab type cap and at least 2 feet thick when used exclusively.

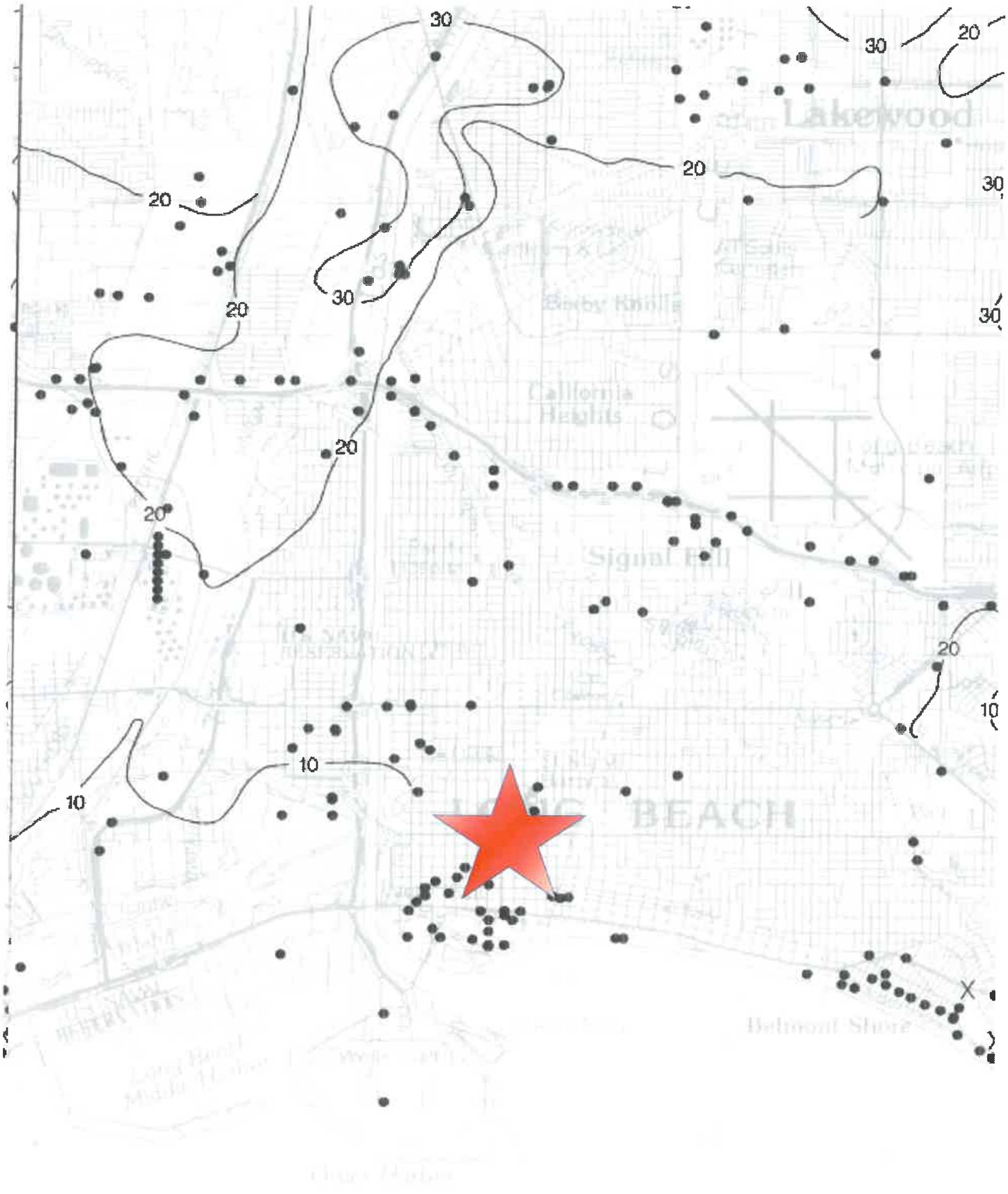

Figure 4: Historical groundwater map (Long Beach Quadrangle)

Any surcharges such as traffic and adjacent building loads shall be computed and adhered into the design by the structural engineer justification. On site percolation should be placed 15 feet away from property line or adjacent public way and any foundation horizontally and vertically.

3.11 Percolation of Storm Water

For the storm water management percolation testing, one 10 feet bore hole was used. Based on a single wall percolation method, the percolation rate was an average of 10 inch per hour without including the safety factor/s. However due to expected historical shallow groundwater at the site on-site percolation is not recommended.

3.12 Observation and Testing

It is recommended that **Soil Pacific Inc.** be present to observe and test during the following stages of construction:

☐ Site grading to confirm proper removal of unsuitable materials and to observe and test the placement of fill.

☐ Inspection of all foundation excavations prior to placement of steel or concrete.

☐During the placement of retaining wall subdrain and backfill materials.

☐ Inspection of all slab-on-grade areas prior to placement of sand, Visqueen.

☐ After trenches have been properly backfilled and compacted.

☐When any unusual conditions are encountered.

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12. Varnes, David J., 1978, Slope Movement and Types and Processes in Landslides: Analysis and Control, Transportation Research Board, National Academy of Sciences, Wash D. C., Sp. Rpt. 176, Chap. 2.

13. Websites
   a. CDMG
   b. USGS. Earthquakes in Southern California
   c. NavigateLA
   d. SCEDC

APPENDIX A

Field Exploration

Log of Sub-surface Exploration

B-1

Std. Pen	Drive Wt:	USCS Letter	Equipment Type: SH2800		Boring # B-1
Bulk/Bag	Drop:	Graphic	Diameter: 4"	Logged by: A.SH.	Date:1/22/18
Ring		Laboratory	Depth: 20 feet	G.water: -	Backfilled:Y

Elev. (feet)	N	Moisture	Dry Reading	USCS Letter	Description of Earth Materials
		13.7	107.2	SM	6 inch concrete slab and subbase. Dark gray, fine grained silty sand with some scattered coarse grained sand, damp and moderately dense, top/fill soils.
5		8.8	108.9	SM	Light brown, brown fine grained silty sand with calich materilas, damp, moderately dense. Native
10		10.2	110.1	SM	Brown to gray fine grained silty sand, and sany layers damp.
15		5.5	112.7	SP	Gray olive fine garined silty sand, damp to dry and moderately dense.
20					
25					End of sub-surface exploration 20 feet.No groundwater or caving observed.
30					
35					
40					

Log depicts conditions at the time and location drilled.

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Project Name: 425 E. 5th Street, Long Beach, California

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Report Date:

Figure:

Log of Sub-surface Exploration

B-2

Equipment Type: SH2800
Boring # B-2
Diameter: 4"
Logged by: A.SH.
Date:1/22/18
Depth: 20 feet
G.water: -
Backfilled:Y

Std. Pen
Bulk/Bag
Ring
Drive Wt:
Drop:
USCS Letter
Graphic
Laboratory

Elev. (feet)	N	Moisture	Dry Reading	USCS Letter	Description of Earth Materials
		12.5	105.4	SM	6 inch concrete slab and subbase. Dark gray, fine grained silty sand with some scattered coarse grained sand, damp and moderately dense, top/fill soils.
5		9.9	109.0	SM	Light brown, brown fine grained silty sand with calich materilas, damp, moderately dense. Native
10				SM	Brown to gray fine grained silty sand, and sany layers damp.
		6.1	110.3		
15				SP	Gray olive fine garined silty sand, damp to dry and moderately dense.
20					
					End of sub-surface exploration 20 feet.No groundwater or caving observed.
25					
30					
35					
40					

Log depicts conditions at the time and location drilled.

Soil Pacific Inc.
Geotechnical and Environmental Services

Project Name: 425 E. 5th Street, Long Beach, California
Project Number: A-6595-17
Report Date:
Figure:

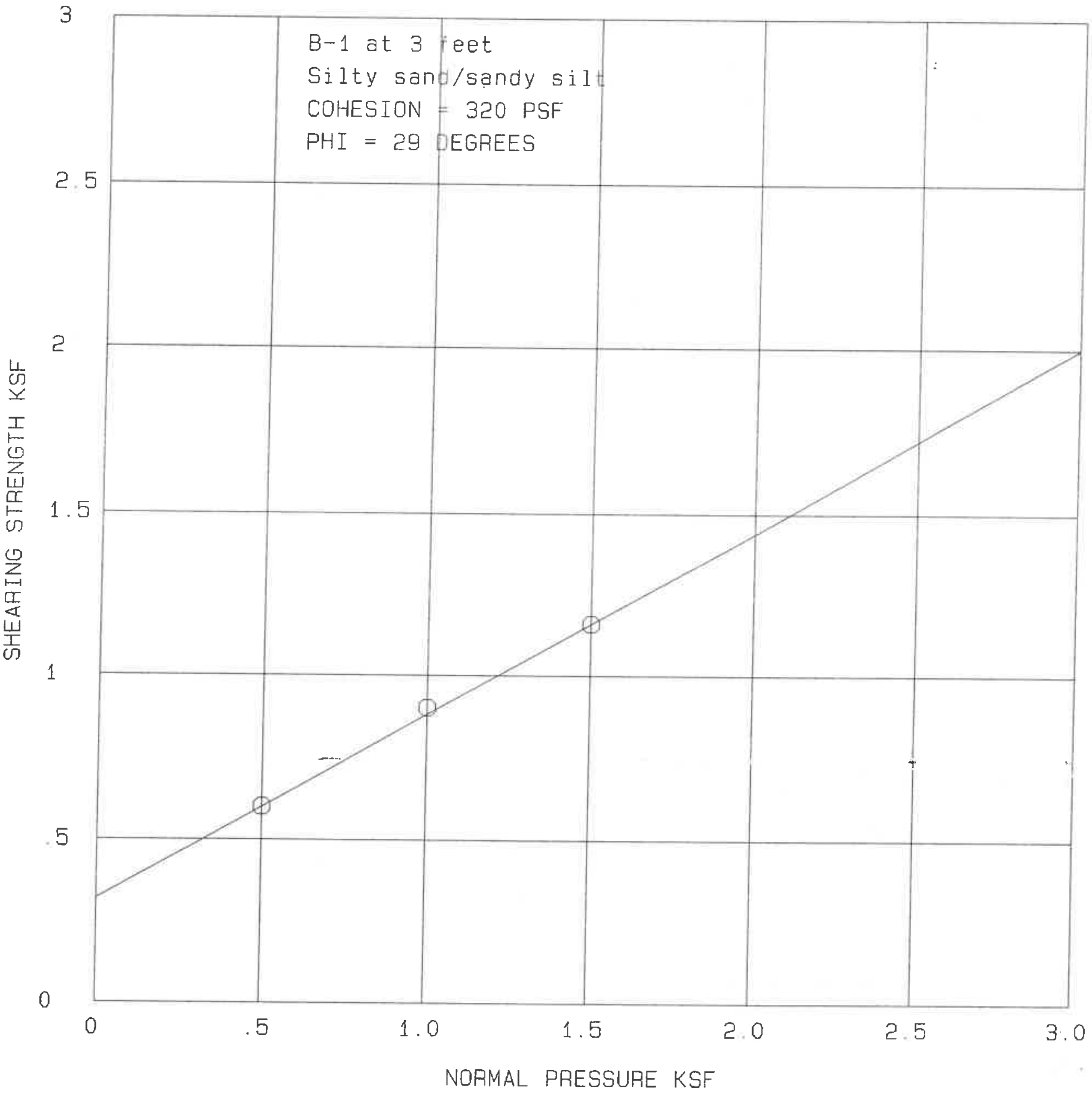
APPENDIX B

Laboratory

SHEAR TEST DIAGRAM

J.O. A-6595-17

DATE 1/22/18



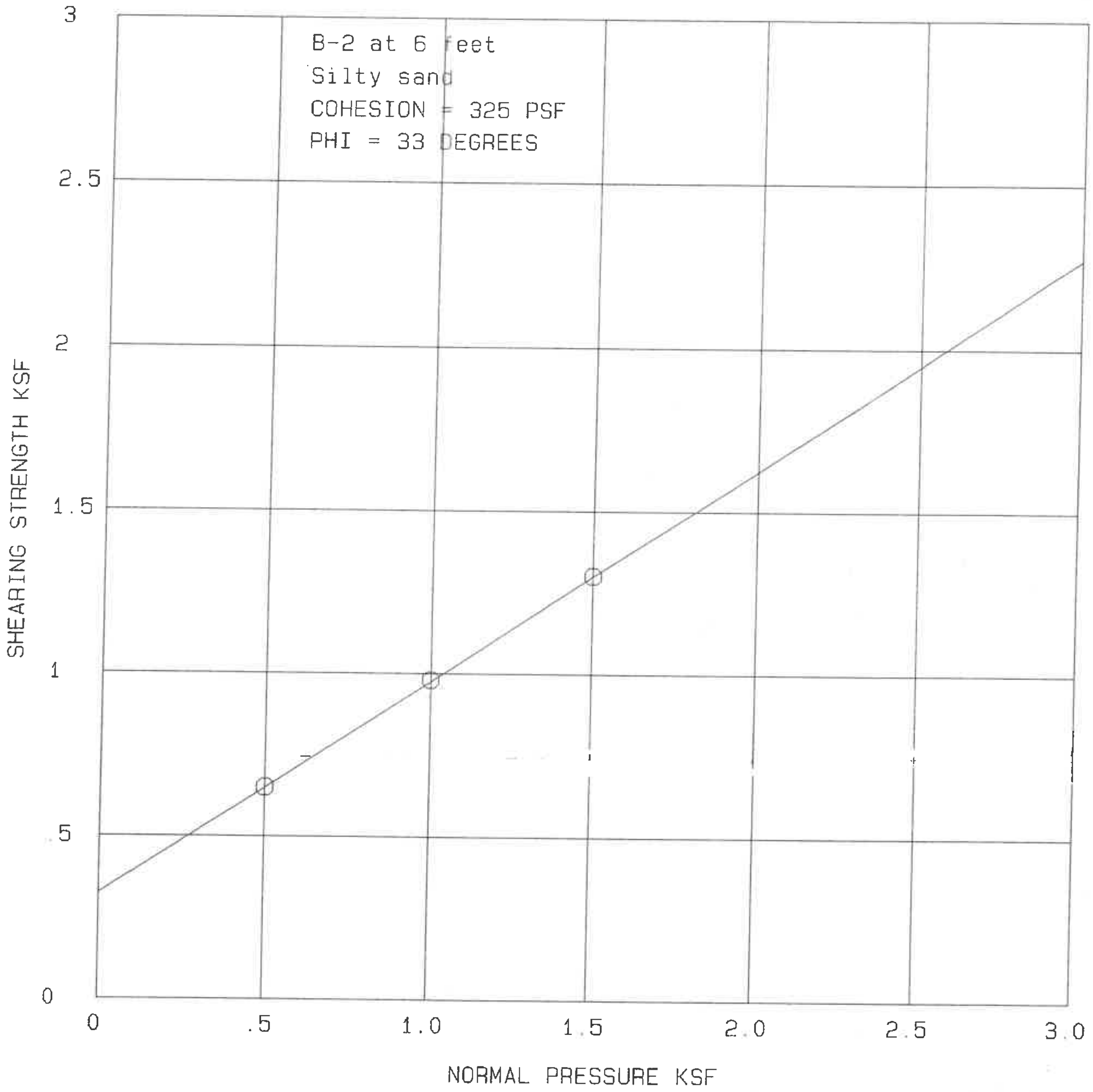
PLATE

APPENDIX

SHEAR TEST DIAGRAM

J.O. A-6595-17

DATE 1/22/18



PLATE

APPENDIX

BEARING VALUE ANALYSIS

J.O. A-6595-17

DATE 1/22/18

COHESION = 320 PSF

GAMA = 120 PCF

PHI = 29 DEGREES

DEPTH OF FOOTING = 2 FEET

BREADTH OF FOOTING = 2 FEET

FOOTING TYPE = SQUARE

BEARING CAPACITY FACTORS

Nc = 27.9

Nq = 16:4

Ng = 15.6

FOOTING COEFFICIENTS

K1 = 1.2

K2 = .4

REFERENCE: TERZAGHI & PECK: 1967, 'SOIL MECHANICS IN ENGINEERING PRACTICE': PAGES 217 TO 225.

FORMULA

ULIMATE BEARING = (K1 * Nc * C) + (K2 * GA * Ng * B) + (Nq * GA * D) = 16145.1

$$\text{ALLOWABLE BEARING} = \frac{\text{ULTIMATE BEARING}}{3} = 5381.7$$

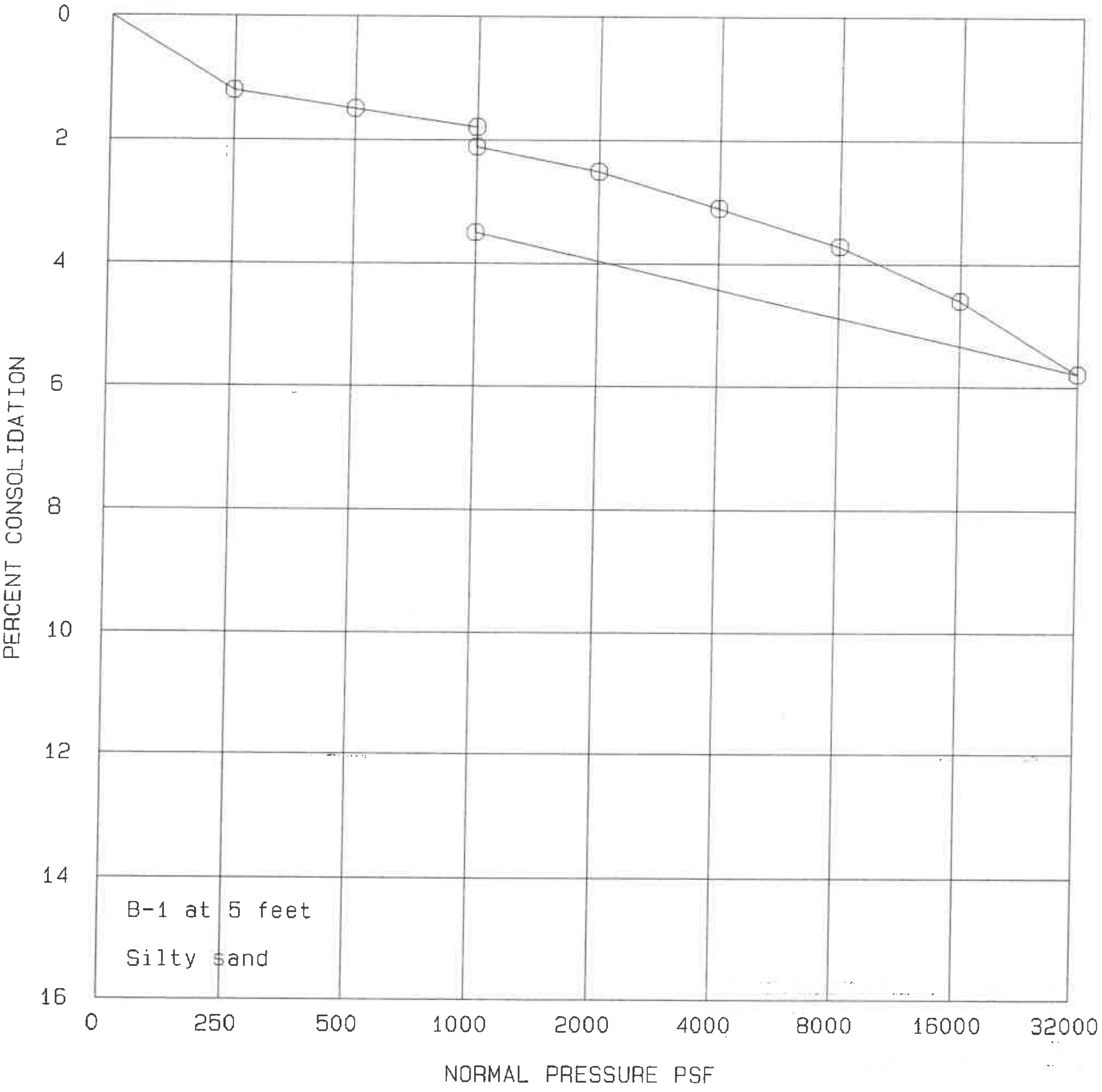
THE ALLOWABLE BEARING VALUE SHOULD NOT EXCEED 5381.7 PSF. DESIGN SHOULD CONSIDER EXPANSION INDEX.

PLATE

CONSOLIDATION PRESSURE CURVE

J.O. A-6595-17

DATE 1/22/18



PLATE

Earth Pressure Calculations

Soil Strength Parameters:

$\phi := 29$

$\gamma := 120$

Active :

$$\mathrm{Ka} := \tan\left[\left(45 - \frac{\phi}{2}\right) \cdot \left(\frac{\pi}{180}\right)\right]^2$$

Active earth Presure $\mathrm{Ka} = 0.347$

$\mathrm{Pa} := \mathrm{Ka} \cdot \gamma$

slope angle range, degrees

$\mathrm{Pa} = 41.637$ LEVEL BACKFILL BEHIND WALL $\mathrm{Pa} = 41.637$

$\mathrm{Pa18} := \mathrm{Pa} \cdot 1.08$ 5:1 BACKFILL BEHIND WALL $\mathrm{Pa18} = 44.968$

$\mathrm{Pa18} := \mathrm{Pa} \cdot 1.22$ 3:1 BACKFILL BEHIND WALL $\mathrm{Pa18} = 50.797$

$\mathrm{Pa39} := \mathrm{Pa} \cdot 1.48$ 2:1 BACKFILL BEHIND WALL $\mathrm{Pa39} = 61.623$

Passive

$$\mathrm{Kp} := \tan\left[\left(45 + \frac{\phi}{2}\right) \cdot \left(\frac{\pi}{180}\right)\right]^2 \qquad \mathrm{Kp} = 2.882$$

Pasive Earth Presure

$\mathrm{Pp} := \mathrm{Kp} \cdot \gamma$

$\mathrm{Pp} = 345.847$

Atrest

$$\mathrm{Kat} := 1 - \sin\left(\phi \cdot \frac{\pi}{180}\right) \qquad \mathrm{Kat} = 0.515$$

$\mathrm{Pat} := \mathrm{Kat} \cdot \gamma$

$\mathrm{Pat} = 61.823$

APPENDIX

TEMPORARY BACKCUT STABILITY

J.O. A-6595-17

DATE 1/22/18

COHESION = 300 PSF GAMA = 120 PCF PHI = 29 DEGREES

CUT HEIGHT = 4 FEET

SOIL TYPE = Silty sand

BACKFILL ASSUMED TO BE LEVEL

PORE PRESSURE NOT CONSIDERED

FORMULA

$$\text{SAFETY FACTOR} = \frac{(C * L) + (GA * AREA * COS(Z) * TAN(PHI))}{GA * AREA * SIN(Z)} = 3.18$$

$$Z = 45 + (PHI/2)$$

SINCE THE SAFETY FACTOR OF 3.18 IS GREATER THAN THE REQUIRED 1.25, THE TEMPORARY EXCAVATION IS CONSIDERED TO BE STABLE. THIS IS WITH A LEVEL AREA EQUAL TO THE LENGTH OF THE VERTICAL CUT ABOVE THE CUT.

PLATE

Analysis of Seismic and Active Earth Pressures on Retaining Walls

Backfill & Wall Parameters

γ =	120	(pcf)	(Soil Unit Weight)
	(Degrees)	**(Radians)**	
Ø =	29.0	0.506	(Soil Internal Friction Angle)
δ =	17.0	0.297	(Wall & Soil Friction Angle)
β =	0.0	0.000	(Slope of Backfill)
α =	0.0	0.000	(Backface Angle of the Wall)

Ground Motion Parameters

S_{DS} =	1.074		Short Period Design Spectral Acceleration (2010 ASCE-7)
PGA =	0.627	(g)	Peak Ground Acceleration (2010 ASCE-7)
K_v =	0		Vertical Seismic Coefficient (Typically set to zero)
K_h =	0.3135		Horizontal Seismic Coefficient

Al Atik & Sitar (2007), Lew et al. (2010)

	PGA <	0.4	(g)	**K_h = 0**
0.4	< PGA <	0.6	(g)	**K_h = 0.25 PGA**
0.6	< PGA <	1.0	(g)	**K_h = 0.50 PGA**
1.0	< PGA		(g)	**K_h = 0.67 PGA**

Mononobe Okabe Active Seismic Pressure Calculation

ψ =	0.304	(radians)		$\tan^{-1}(k_h/1\text{-}k_v)$
K_{AE} =	**0.605**			Coefficient of Seismic Active Earth Pressure
P_{AE} =	**72.6**	**1/2 H^2**	(lb/ft)	Seismic Active Earth Pressure

$$K_{AE} = \frac{Cos^2\,(\varnothing - \delta - \alpha)}{Cos\,(\psi)\,Cos^2\,(\alpha)\,Cos\,(\delta + \alpha + \psi)\left[1 + \sqrt{\dfrac{Sin\,(\varnothing + \delta)\;Sin\,(\varnothing - \beta - \psi)}{Cos\,(\delta + \alpha + \psi)\;Cos\,(\beta - \alpha)}}\right]^2}$$

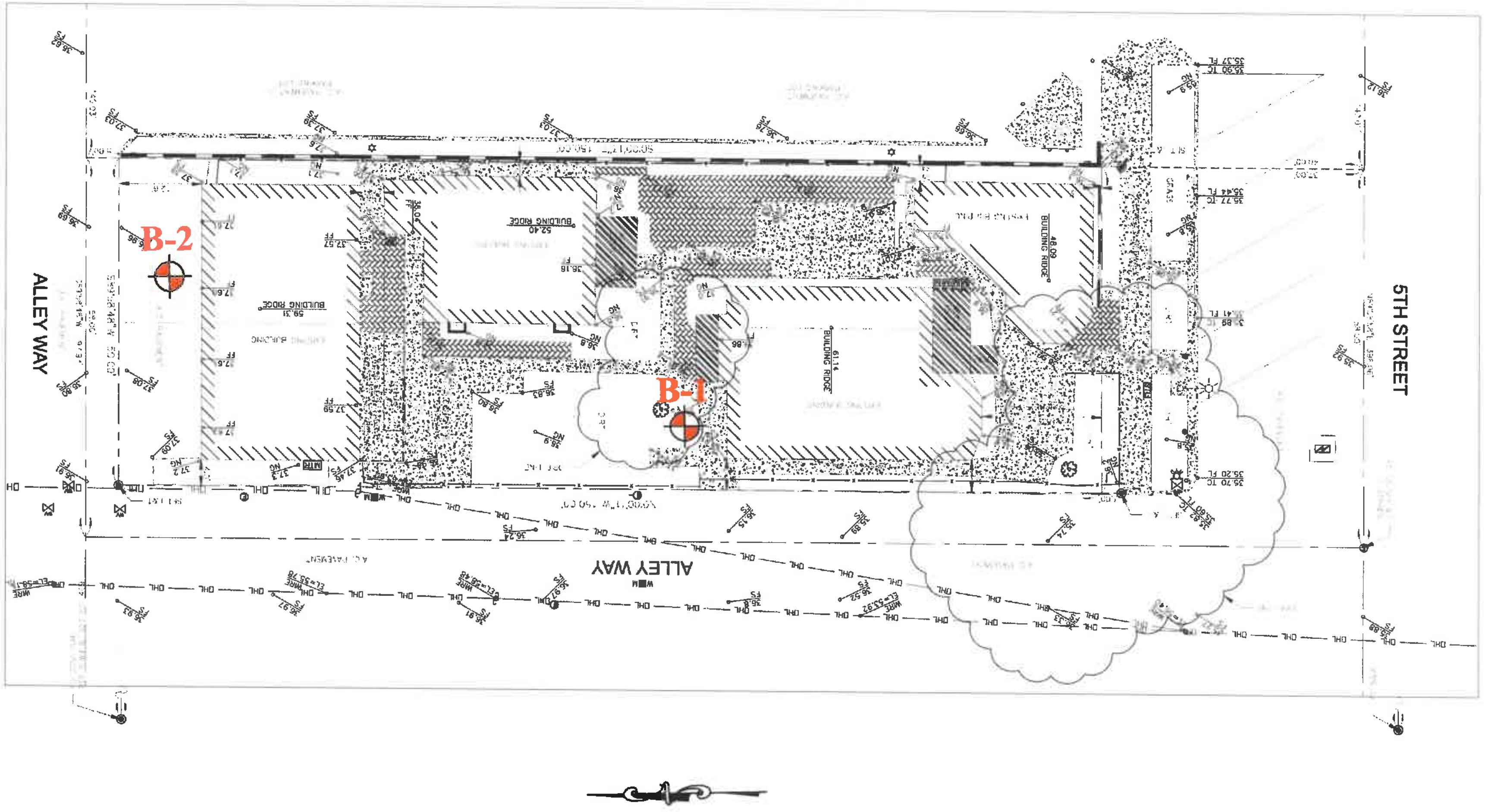
Coulomb's Active Earth Pressure Calculation

K_A =	**0.311**			Coefficient of Active Earth Pressure
P_A =	**37.3**	**1/2 H^2**	(lb/ft)	Active Earth Pressure
$\Delta\ K_{AE}$ =	**K_{AE} - K_A**			Coefficient of Seismic Earth Pressure
$\Delta\ K_{AE}$ =	**0.294**			
$\Delta\ P_{AE}$ =	**35.3**	**1/2 H^2**	(lb/ft)	Seismic Earth Pressure

$$K_A = \frac{Cos^2\,(\varnothing - \alpha)}{Cos^2\,(\alpha)\,Cos\,(\delta + \alpha)\left[1 + \sqrt{\dfrac{Sin\,(\varnothing + \delta)\;Sin\,(\varnothing - \beta)}{Cos\,(\delta + \alpha)\;Cos\,(\beta - \alpha)}}\right]^2}$$

APPENDIX C

References



LEGEND

⊕ Boring Location

soil PACIFIC Inc.
Geotechnical & Environmental Services
675 N. Eckhoff, Suite # A
Orange, CA 92868

Project Location:
425 E. 5TH., Long Beach, CA

PLOT PLAN

FIGURE-A-1-1

PROJECT NO.: A-6595-18

DATE : 01/25/2018

SHEET 1 OF 1

USGS Design Maps Summary Report

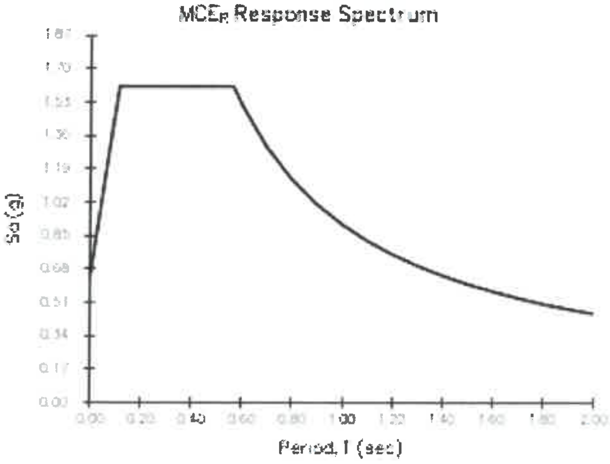
User–Specified Input

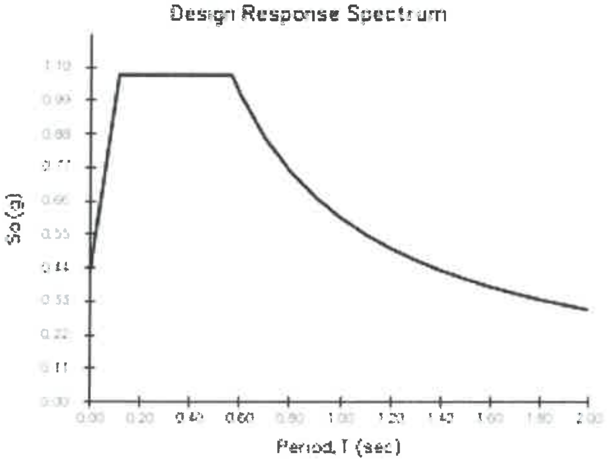
Report Title	A-6595-18 Thu January 25, 2018 19:44:20 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.7733°N, 118.18692°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S =	1.611 g	S_{MS} =	1.611 g	S_{DS} =	1.074 g
S_1 =	0.605 g	S_{M1} =	0.908 g	S_{D1} =	0.605 g

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


MCE$_R$ Response Spectrum


Design Response Spectrum

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

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.7733°N, 118.18692°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 1.611 g
From Figure 22-2 [2]	S_1 = 0.605 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.611 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	$\boxed{1.5}$
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.605 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.611 = 1.611$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 0.605 = 0.908$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

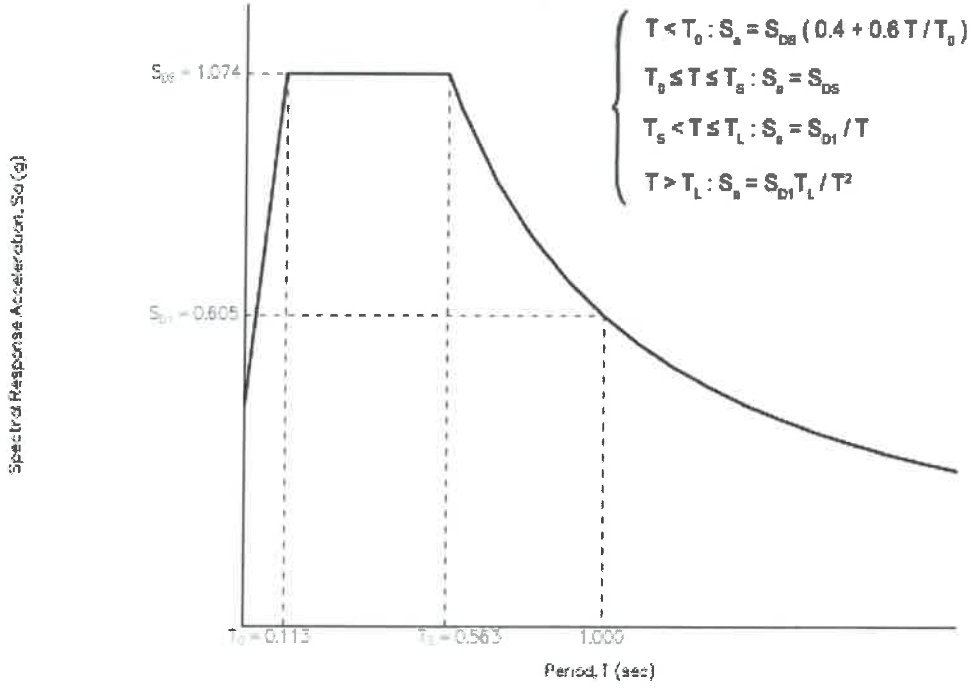
Equation (11.4–3): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.611 = 1.074$ g

Equation (11.4–4): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.908 = 0.605$ g

Section 11.4.5 — Design Response Spectrum

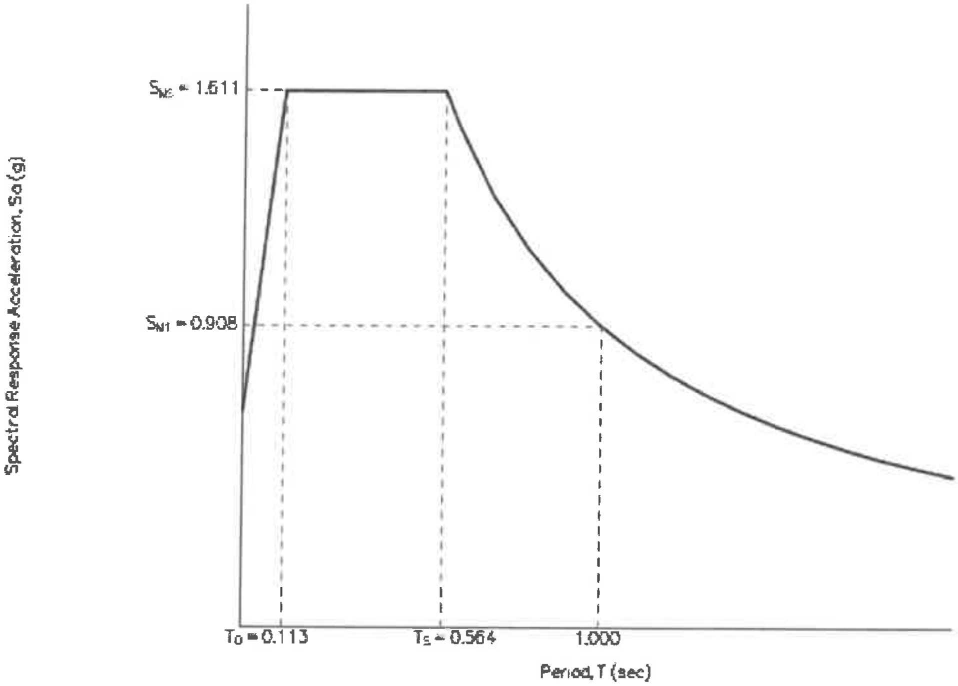
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.627

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.627 = 0.627$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.627 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] $C_{RS} = 0.944$

From Figure 22-18 [6] $C_{R1} = 0.954$

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.074 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.605 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX D

General Grading Specifications

GENERAL EARTHWORK
AND GRADING SPECIFICATIONS

1. GENERAL INTENT

These specifications present general procedures and requirements for grading and earthwork as shown on the approved grading plans, including preparation of areas to be filled, placement of fill, installation of subdrains, and excavations. The recommendations contained in the geotechnical report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict. Evaluations performed by the consultant during the course of grading may result in new recommendations of the geotechnical report.

2.EARTHWORK OBSERVATION AND TESTING

Prior to the commencement of grading, a qualified geotechnical consultant (soils engineer and engineering geologist, and their representatives) shall be employed for the purpose of observing earthwork and testing the fills for conformance with the recommendations of the geotechnical report and these specifications. It will be necessary that the consultant provide adequate testing and observation so that he may determine that the work was accomplished as specified. It shall be the responsibility of the contractor to assist the consultant and keep him apprised of work schedules and changes so that he may schedule his personnel accordingly.

It shall be the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If in the opinion of the consultant, unsatisfactory conditions, such as questionable soil, poor moisture condition, inadequate compaction, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the consultant will be empowered to reject the work and recommend that construction be topped until the conditions are rectified. Maximum dry density tests used to determine the degree of compaction will be performed in accordance with the American Society of Testing and Materials tests method ASTM D 1557-00.

3.0 PREPARATION OF AREAS TO BE FILLED

3.1 Clearing and Grubbing: All brush, vegetation and debris shall be removed or piled and otherwise disposed of.

3.2 Processing: The existing ground which is determined to be satisfactory for support of fill shall be scarified to a minimum depth of 6 inches. Existing ground which is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until the soils are broken down and free of large clay lumps or clods and until the working surface is reasonably uniform and free of uneven features which would inhibit uniform compaction.

3.3 Overexcavation: Soft, dry, spongy, highly fractured or otherwise unsuitable ground, extending to such a depth that the surface processing cannot adequately improve the condition, shall be overexcavated down to firm ground, approved by the consultant.

3.4 Moisture Conditioning: Overexcavated and processed soils shall be watered, dried-back, blended, and/or mixed, as required to attain a uniform moisture content near optimum.

3.5 Recompaction: Overexcavated and processed soils which have been properly mixed and moisture- conditioned shall be recompacted to a minimum relative compaction of 90 percent.

3.6 Benching: Where fills are to be placed on ground with slopes steeper than 5: 1 (horizontal to vertical units), the ground shall be stepped or benched. The lowest bench shall be a minimum of 15 feet wide, shall be at least 2 feet deep, shall expose firm material, and shall be approved by the consultant. Other benches shall be excavated in firm material for a minimum width of 4 feet. Ground sloping flatter than 5 : 1 shall be benched or otherwise overexcavated when considered necessary by the consultant.

3.7 Approval: All areas to receive fill, including processed areas, removal areas and toe-of-fill benches shall be approved by the consultant prior to fill placement.

4.0 FILL MATERIAL

4.1 General: Material to be placed as fill shall be free of organic matter and other deleterious substances, and shall be approved by the consultant. Soils of poor gradation, expansion, or strength characteristics shall be placed in areas designated by consultant or shall be mixed with other soils to serve as satisfactory fill material.

4.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches, shall not be buried or placed in fills, unless the location, materials, and disposal methods are specifically approved by the consultant. Oversize disposal operations shall be such that nesting of oversize material does not occur, and such that the oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 feet vertically of finish grade or within the range of future utilities or underground construction, unless specifically approved by the consultant.

4.3 Import: If importing of fill material is required for grading, the import material shall meet the requirements of Section 4. 1.

5.0 FILL PLACEMENT AND COMPACTION

5.1 Fill Lifts: Approved fill material shall be placed in areas prepared to receive fill in near-horizontal layers not exceeding 6 inches in compacted thickness. The consultant may approve thicker lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to attain uniformity of material and moisture in each layer.

5.2 Fill Moisture: Fill layers at a moisture content less than optimum shall be watered and mixed, and wet fill layers shall be aerated by scarification or shall be blended with drier material. Moisture-conditioning and mixing of fill layers shall continue until the fill material is at a uniform moisture content or near optimum.

5.3 Compaction of Fill: After each layer has been evenly spread, moisture conditioned, and mixed, it shall be uniformly compacted to not less than 90 percent of maximum dry density. Compaction equipment shall be adequately sized and shall be either specifically designed for soil compaction or of proven reliability, to efficiently achieve the specified degree of compaction.

5.4 Fill Slopes: Compaction of slopes shall be accomplished, in addition to normal compacting procedures, by backfilling of slopes with sheepsfoot rollers at frequent increments of 2 to 3 feet in fill elevation gain, or by other methods producing satisfactory results. At the completion of grading, the relative compaction of the slope out to the slope face shall be at least 90 percent.

5.5 Compaction Testing: Field tests to check the fill moisture and degree of compaction will be performed by the consultant. The location and frequency of tests shall be at the consultant's discretion. In general, the tests will be taken at an interval not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of embankment.

6.0 SUBDRAIN INSTALLATION

Subdrain systems, if required, shall be installed in approved ground to conform to the approximate alignment and details shown on the plans or herein. The subdrain location or materials shall not be changed or modified without the approval of the consultant. The consultant, however, may recommend and upon approval, direct changes in subdrain line, grade or material. All subdrains should be surveyed for line and grade after installation, and sufficient time shall be allowed for the surveys, prior to commencement of filling over the subdrains.

7.0 EXCAVATION

Excavation and cut slopes will be examined during grading. If directed by the consultant, further excavation or overexcavation and refilling of cut areas shall be performed, and/or remedial grading of cut slopes shall be performed. Where fill-over-cut slopes are to be graded, unless otherwise approved, the cut portion of the slope shall made and approved by the consultant prior to placement of materials for construction of the fill portion of the slope.

8.0 TRENCH BACKFILLS

8.1 Supervision: Trench excavations for the utility pipes shall be backfilled under engineering supervision.

8.2 Pipe Zone: After the utility pipe has been laid, the space under and around the pipe shall be backfilled with clean sand or approved granular soil to a depth of at least one foot over the top of the pipe. The sand backfill shall be uniformly jetted into place before the controlled backfill is placed over the sand.

8.3 Fill Placement:'The onsite materials, or other soils approved by the engineer, shall be watered and mixed as necessary prior to placement in lifts over the sand backfill.

8.4 Compaction: The controlled backfill shall be compacted to at least 90 percent of the maximum laboratory density as determined by the ASTM compaction method described above.

8.5 Observation and 'Testing: Field density tests and inspection of the backfill procedures shall be made by the soil engineer during backfilling to see that the proper moisture content and uniform compaction is being maintained. The contractor shall provide test holes and exploratory pits as required by the soil engineer to enable sampling and testing.