Project No. 1855
3 March 2014

WAYNE TING & ASSOCIATES, INC.

GEOTECHNICAL CONSULTANTS
42329 Osgood Road, Unit A, Fremont, CA 94539
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tingwayne@yahoo.com

Mr. Hardy Gill
c/o Greenwood & Moore, Inc.
3111 Castro Valley Boulevard, Suite 200
Castro Valley, CA 94546

Subject: **UNCONTROLLED FILLS**
Proposed Single-Family Subdivision
Tract No. 8022
2512 and 2492 D Street
Alameda County, California

References:
1) Geotechnical Report Review
By Wayne Ting & Associates, Inc.
Dated 7 May 2013
2) Geologic Investigation
By Buckley Engineering Associates, Dated 21 August 2002
3) Uncontrolled Fill Investigation
By Wayne Ting & Associates, Inc.
Dated 5 August 2010
4) Report of Testing and Observation Service During Backfill The New Culvert
By Wayne Ting & Associates, Inc.
Dated 27 May 2010
5) Removal of Uncontrolled Fills
By Wayne Ting & Associates, Inc.
Dated 8 November 2010
6) State of California Seismic Hazard Zones, Hayward Quadrangle Official Map, Released date: July 3, 2003

Dear Mr. Gill:

At your request, Wayne Ting & Associates, Inc. (WTAI) has reviewed the referenced materials and performed a site reconnaissance and excavated eleven test pits on March 3, 2014 to locate the existing uncontrolled fills at the subject project. The approximate locations of uncontrolled fills shown as dot circles and excavated test pits shown as solid dots are shown the map, Figure 1, Appendix A. The test pit descriptions are provided as follows:

Test Pit 1: the upper 6 feet are uncontrolled fills consisting of brown sandy clay with rock fragments, concrete debris, plasticity paper, and glass, followed by native brown sandstone to maximum depth excavated to 6.5 feet.

Test Pit 2: the upper 4 feet are uncontrolled fills consisting of dark brown sandy clay with rock fragments, wood, plasticity paper, and broken clay pipe, followed by native brown sandstone to maximum depth excavated to 4.5 feet.

Test Pit 3: the upper 6 to 12 inches are brown native sandy clays, followed by brown sandstone to maximum depth excavated to 1.5 feet.

Test Pit 4: the upper 6 inches are native brown sandy clay, followed by brown sandstone to maximum depth excavated to 3.0 feet.

Test Pit 5: the upper 18 inches are native brown sandy clay, followed by brown sandstone to maximum depth excavated to 2.0 feet.

Test Pit 6: the upper 36 inches are native medium brown sandy clay, followed by brown sandstone to maximum depth excavated to 4.0 feet.

Test Pit 7: the upper 8.0 feet are uncontrolled fills consisting of brown sandy clay with rock fragments and asphaltic concrete debris, followed by native dark brown silty clay to maximum depth excavated to 9.0 feet.

Test Pit 8: the upper 5.0 feet are uncontrolled fills consisting of brown sandy clay with rock fragments and crushed to maximum depth excavated to 5.0 feet.

Test Pit 9: the upper 5.0 feet are uncontrolled fills consisting of brown sandy clay with rock fragments, asphaltic concrete, and plasticity paper, followed by native dark gray silty clay to maximum depth excavated to 5.5 feet. Water seepage was observed at 5.0 feet below the ground surface.

Test Pit 10: the upper 2.0 feet are uncontrolled fills consisting of dark brown and brown sandy clay, followed by brown sandstone to maximum depth excavated to 2.5 feet.

Test Pit 11: the upper 5.0 feet are uncontrolled fills consisting of dark brown and light brown sandy clay mixtures, followed by light brown sandy clay to the maximum depth excavated to 2.5 feet.

Based on our site reconnaissance, review of Reference 5 and the above described test pit logs, it is our opinion that uncontrolled fills founded on west existing a dirt road in Reference 3 had been removed from the site. It is noted native sandstone was observed on the surface of a slope at the location of boring 1 in reference 3. Therefore, I assume that location of boring 1 in reference 3 may be not marked correctly. In addition, in Reference 4, engineered fills have been constructed at the existing south culvert and tested by us. It is noted that during our field investigation, existing uncontrolled fills were encountered around the existing north culvert and proposed lots 5, 6, 8 and 9. The uncontrolled fills were also encountered at the proposed lot 12. In addition, loose fills were backfilled in the seven test pits during the geologic investigation (Reference 2.) The locations of the test pits are shown Figure 1.

It is noted that recommendation of removal and recompaction the uncontrolled fills are provided in page 3 and 4 of Reference 1. However, if there are any uncontrolled fills except the above-mentioned areas are encountered during the site grading, these fills should be removed and recompacted.

It is noted that both sides of the south creek in the proposed lot 12 are located within the potential landslide induced by earthquake geologic hazards zones. Therefore, storm water should not be discharged to these areas to reduce the stability of these slopes. The locations of these hazard zones are located on the Figure 1.

Should you have any questions relating to the contents of this report, please contact our office at your convenience.

Very truly yours,

WAYNE TING & ASSOCIATES, INC.

Wayne L. Ting, C.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
WAYNE L. TING
No. C 46276
Exp: 12/31/14
CIVIL
STATE OF CALIFORNIA

Copies: 2 to Mr. Gill

Location of Uncontrolled Fills

Potential Landslide Induced by Earthquake Geologic Hazard Zone

Test Pit No.	Depths of Uncontrolled Fills
1	6.0 feet
2	4.0 feet
3	None
4	None
5	None
6	None
7	8.0 feet
8	5.0 feet
9	5.0 feet
10	2.0 feet
11	5.0 feet

WAYNE TING & ASSOCIATES, INC.	***Site Plan***	*Figure No. 1*
GEOTECHNICAL CONSULTANTS	Scale: 1"=100'	*Page No. 4*

2013.007

WAYNE TING & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

Project No. 1855
7 May 2013

Mr. Hardy Gill
Shaw Group, LP
P. O. Box 2622
Sumas, WA 98295

Subject: **GEOTECHNICAL REPORT REVIEW**
Proposed Single-Family Subdivision
Tract No. 8022
2512 and 2492 D Street
Alameda County, California

References:
1) Geotechnical Investigation
By Cleary Consultants, Inc.
Dated 7 July 1989
2) Geologic Investigation
By Buckley Engineering Associates, Dated 21 August 2002
3) Update of Geotechnical Investigation and Supplemental Recommendations
By Wayne Ting & Associates, Inc.
Dated 1 April 2010
4) Uncontrolled Fill Investigation
By Wayne Ting & Associates, Inc.
Dated 5 August 2010
5) Proposed Subdivision
By Wayne Ting & Associates, Inc.
Dated 16 January 2006

Dear Mr. Gill:

At your request, Wayne Ting & Associates, Inc. (WTAI) has reviewed the referenced materials to provide geotechnical recommendations for the design and construction of the subject project. The tentative map is provided in Figure 1, Appendix A.

It is noted that site plan, test pit logs, and boring logs obtained from References 2 and 4 are provided in Appendix B. In addition, it is noted that the uncontrolled fills mentioned in Reference 4 have been removed to the native soils.

Addendum Attachment E-2/p.1

EARTHQUAKE-INDUCED LANDSLIDE ANALYSIS

Background

It is noted that the proposed subject site consisted of moderate to 2:1 (horizontal:vertical) slopes. Detail site descriptions are provided in the referenced reports. The subject site is located within the earthquake-induced landslide zones based on the California Seismic Hazard Zones, Hayward Quadrangle map, dated July 2, 2003, the proposed development will need to address the potential of permanent ground displacement during earthquakes. Our evaluation is based on California Department of Conservation, Division of Mines and Geology 's Special Publication 117A (SP 117), Guidelines for Evaluation and Mitigating Seismic Hazards in California. We conducted seismic slope stability analysis that is consistent with the "Recommended Procedure for Implementation of DMG Special Publication 117A Guidelines for Analyzing and Mitigating Landslide Hazards in California," developed by the ASCE Implementation Committee, chaired by Thomas F. Blake, dated June 2002 (Blake et al 2002).

The results of analysis based on the following geotechnical parameters were presented in References 3 and 4. The detail analysis and printout are not provided in this report.

Soil and Rock Geotechnical Parameters

The laboratory test results, our field observations and engineering experience form the basis for using the following engineering properties in our stability analysis:

Material	Unit Weight (p.c.f.)	Cohesion (p.s.f.)	Friction Angle (degrees)	Case No.
Silty clay (native)	120	540	16.0	1
Silty clay (Recommended by Cal Engineering)	120	250	25.0	2
Sandstone	130	1,000	35	

Stability Analysis Results

The results of the stability analysis are summarized as follows:

Failure Plane	Loading Condition	Pseudo Static Factor of Safety	Case No.
Circular	Undrained Strength	1.92	1
Circular	Undrained Strength	1.89	2

A factor of safety of 1.2 or greater for the pseudo-static analyses is considered to be adequate. The result of the pseudo-static factor of safety at the subject site is average of 1.9. Therefore, the analysis indicates the existing slopes meet the minimum factor of safety criteria stated in SP 117A. It is our opinion that permanent ground deformation during strong earthquakes would be small, if any.

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

GENERAL CONSIDERATIONS

1. From a geotechnical engineering standpoint, it is the opinion of WTAI that the subject site is suitable for the proposed construction provided the project design and construction incorporate the recommendations contained herein.

2. It is recommended that the WTAI be given the opportunity to review the grading and foundation plans and specifications when completed, to evaluate compliance with the recommendations provided in this report.

3. It is further recommended that WTAI be retained for testing and observation during all grading and foundation construction phases to help determine that the design requirements are fulfilled. WTAI should be notified at least 48 hours prior to grading and/or foundation operations on this project.

4. Any work related to the grading and/or foundation operations performed without the direct observation of WTAI will invalidate the recommendations of this report.

SITE PREPARATION AND GRADING

5. Prior to grading, the proposed structure, pavement, and fill areas should be cleared of all obstructions and deleterious materials. It is noted that the test pits mentioned in Reference 2 were loosely backfilled. Therefore, these loose fill in these pits and any uncontrolled fills should be overexcavated and backfilled with engineered fills and compacted to not less than 95 percent relative compaction.

6. After clearing, these areas should be stripped of all organic topsoil. It is estimated that stripping depths of 4 to 6 inches may be necessary. However, final stripping depths should be determined by WTAI in the field. The predominantly organic material from the stripping should be removed from the site.

7. After completion of the stripping, the top 8 inches of exposed native ground should be scarified. After scarifying, it should be disced or bladed until it is uniform and free of large clods. The exposed native subgrade soils will be watered or aerated as necessary to bring the soils to a moisture content of 3 percent above the optimum moisture amount. The subgrade should then be uniformly recompacted to a minimum degree of relative compaction of 90 percent of the maximum dry density as determined by ASTM D1557 Latest Edition Laboratory Test Procedure. Materials generated from the excavation may be used as engineered fill with the approval of WTAI provided they are not contaminated by debris.

8. Following recompaction of the native subgrade soils, the site may be filled to the desired finished grade using suitable on-site native soil. All fills should be placed in lifts not exceeding 8 inches in uncompacted thickness and compacted to the abovementioned compaction requirements. Each layer will be spread evenly and will be blade mixed thoroughly to provide uniformity of soil in each layer. Compaction of each layer will be continuous over the fill area and continued until the required density is obtained.

9. Cut and fill transition at garage concrete slabs-on-grade area may experience abrupt differential settlement causing significant distress. This condition can be mitigated by scarifying the cut portion of the transition garage pad a minimum depth of 12 inches. The scarified material should be properly moisture-conditioned to at least 2 percent above optimum moisture content and be recompacted to a minimum relative compaction of 90 percent. It is noted that a minimum three feet of uniform engineered fill should be constructed under the entire garage area. The fill should be placed in thin lifts not exceeding 8 inches in uncompacted thickness and compacted to the abovementioned compaction requirements.

SLOPES

10. In general, all fill slopes should not be steeper than 2:1 (horizontal:vertical). Cut slopes in stiff natural materials should not exceed 2:1 (H:V).

11. A shear key must be established at the toe of all fill slopes where the natural hill slope exceeds 6:1 (horizontal: vertical). The shear key must be at least 12 feet in width and 3 feet cut into the

underlying rock. The bottom of the keyway excavation should be sloping back into the hillside at a minimum gradient of 5 percent. The location and depth of the keyway and subdrain should be determined by WTAI during grading operations. Subsequent benches should be placed at vertical heights of 3 feet and should extend horizontally into the rock. A typical section is presented in Figure 2, Fill Slope Detail.

12. During the grading operations, fill slopes must be compacted and should be over-constructed. At the completion of grading operations, the excess fill or loose soils existing on the slopes should be cut to a firm and adequately designed slope grade. Track-walking of the slope surface should only be utilized to seal the surface.

13. Before work is stopped due to heavy rains, a positive gradient away from slopes should be provided to carry surface runoff water away from the slope and to areas where erosion can be controlled. After the completion of slope grading, the exposed cut and fill slopes should be planted with deep-rooted native plants to minimize erosion. Some minor erosion on slopes should be expected. Thus, periodic maintenance is required.

CALIFORNIA BUILDING CODE SITE CHARACTERIZATION

14. The following design values are base on the geologic information, longitude and latitude of the site and the USGS computer program (2007). Furthermore, in according to Chapter 16 of the 2010 California Building Code (CBC), the site seismic design values have been provided as follows:

CBC Category/Coefficient	Design Value
Figure 1613.5.(3), Short-Period MCE at 0.2s, Site Class B, Ss	1.875
Figure 1613.5.(4), 1.0s Period MCE, Site Class B, S1	0.712
Table 1613.5.2, Soil Profile Type, Site Class	D
Table 1613.5.3(1), Site Coefficient, Fa	1.0
Table 1613.5.3(2), Site Coefficient, Fv	1.5
S_{MS} = Fa x S_s Spectral Response Accelerations	1.875
S_{M1} = Fv x S_1 Spectral Response Accelerations	1.068
S_{DS} = 2/3 x S_{MS} Design Spectral Response Accelerations	1.250
S_{D1} = 2/3 x S_{M1} Design Spectral Response Accelerations	0.712

** Latitude 37.6797, Longitude: -122.05624

FOUNDATIONS

15. The drilled piers should have a minimum diameter of 16 inches and a minimum embedment of a minimum 10.0 feet into rock. These piers should be designed for an allowable skin friction value of 500 pounds per square foot for dead plus live loads. This value can be increased by one-third for total loads which include wind or seismic forces. This value is only applicable for piers are penetrating into rock. The validity of this value is based on a minimum pier spacing of 3 pier diameters measured center-to-center. In addition, piers should be tied together with the tie beams.

16. Due to the slope gradient and the expansive soil, any piers located near or on the slope may be subject to creep loads imposed by the soils. For all piers constructed at or within 10 feet from the top of the slope, a triangular pressure distribution of 65 p.c.f. equivalent fluid weight should be designed against the side of these piers along the length in the upper 4 feet of the piers.

17. Resistance to lateral force may be provided by passive earth pressure mobilized along the pier length below the depth of 4 feet. Passive earth pressure may be computed as an equivalent fluid weighing of 300 p.c.f. For design of isolated piers, the allowable passive pressure may be increased by a factor of 1.5.

18. After the pier drilling has completed, the bottom of the pier holes should be cleaned of excessive loose materials prior to placing the reinforcing steel and concrete.

19. Depressions at the top of piers resulting from drilling operations should be backfilled to prevent ponding of water. Care should be exercised during concrete placement to prevent the concrete from spilling around the pier shafts. If excess spillage occurs, the fresh concrete should be removed.

20. Difficult drilling may be encountered in the dense rock. Heavy duty drilling equipment should therefore be used to drill the pier holes.

RETAINING WALL

21. The following design parameter should be used for structural design of proposed retaining walls at the subject site. The drainage detail behind the wall is provided in Figure 3.

TABLE I

Slope Inclination Behind Wall (Horizontal : Vertical)	Equivalent Fluid Weight (Pounds Per Cubic Foot)	
	Unrestrained	Restrained
Flat	45	65
2:1	65	85

In addition, earthquakes induced lateral loads should be added for the basement wall design. These lateral loads should be taken as that imposed by an equivalent fluid weight of 30 p.c.f. However, the distribution of this load should be considered as a triangle with resultant force acting at a point 0.6 of the wall height above the base of the wall.

22. The above criterion is based upon a sufficient drainage system to be constructed behind the walls to prevent the build-up of hydrostatic pressures. The wall drainage system should consist of a gravel blanket with a minimum width of 12 inches and should extend vertically to 12 inches below the ground surface. The top 12 inches should be backfilled with on-site soils to provide a surface seal and be graded away from the wall. If the excavated area behind the wall exceeds 12 inches, the entire excavated space behind the 12-inch blanket material should be backfilled with gravel. The gravel blanket may consist of crushed rock wrapped effectively with filter fabric.

23. A 4-inch diameter perforated pipe should be placed on bedding at the bottom of the gravel blanket adjacent to the base of the footing or grade beam. The perforations should be placed facing down toward bottom of the excavation. The bedding material should be at least 4 inches thick. The pipe should have a minimum gradient of 1.0 percent and should connect to an adequately controlled outlet facility away from the foundations.

CONCRETE SLABS ON GRADE

24. To reduce the potential cracking of the concrete slabs, the following recommendations are made:

a. Slabs-on-grade in the garage area should be reinforced by the structural engineer and should not be doweled into the perimeter foundation.

b. Slabs at garage door openings should be constructed with a thickened edge extending a minimum of 8 inches into the native ground or compacted fill.

c. Concrete slab-on-grade should be underlain by at least 4 inches of clean crushed, 3/4-inch size rock, to act as a cushion and capillary break between the subsoil and the slab.

TRENCH BACKFILL

25. Backfilling and compaction of utility trenches must meet the requirements published by the County of Alameda, Department of Public Works. All trench backfill under pavement areas must be backfilled with baserock or imported granular materials and compacted to at least 90% relative compaction as determined by ASTM D1557 Latest Edition Laboratory Test Procedure. The top 12 inches of the subgrade should be compacted to 95%.

26. Backfill of utility trenches extending under the building area should be properly compacted to ensure against water migration underneath the foundation structure.

PAVEMENT SECTION

27. The top 10 inches of street subgrade should be scarified and recompacted to a minimum relative compaction of 95% and at 2% above the optimum moisture content as determined by ASTM D1557 Latest Edition Laboratory Test Procedure.

28. Aggregate subbase should then be placed on top of the subgrade and compacted to a minimum relative compaction of 95%. Class II aggregate base must also be compacted to 95% relative compaction. The class II aggregate base should conform to the requirements of Standard Specifications of Caltrans, Section 26-1.02A.

29. Pavement Sections: The following recommended pavement sections are based on Traffic Indices (T.I.) of 4, 5 and 6, and assuming R-value of 5.

Traffic Index	Asphaltic Concrete	Class II Aggregate	Aggregate Subbase
4	3.0"	8.0"	11.0"
5	3.0"	12.0"	15.0"
6	4.0"	13.0"	17.0"

DRAINAGE

30. A foundation drain system should be constructed around the perimeter foundations. The foundation drain should be constructed at a lateral distance of 6.0 inches from the foundation and extended a minimum depth of 18 inches below the bottom of the grade beam. The recommended subdrain detail is presented in Figure 3. The perforated pipe shown in Figure 4 will pass into a solid line pipe at the end drain then be directed to a suitable discharge area. Cleanout risers should be provided at the upgradient end of the perforated pipe, at sharp bends, and at 100 foot maximum intervals.

31. All downspouts from the roof gutter system should be tied into a closed pipe system and discharged to an adequate drainage system.

32. Exterior flatwork should be sloping away from the building so that water will be drained away from the structure. Landscape mounds or concrete flatwork should not be constructed to block or obstruct the surface drainage measures.

33. Planted areas should be avoided immediately adjacent to the structure. If planting adjacent to the residence is desired, use of plants that require little moisture is recommended. Sprinkler systems should not be installed where they may cause ponding or saturation of foundation soils. Such ponding or saturation could result in undesirable soil movement, loss of compaction, and/or subsequent foundation and slab movement. Irrigation of landscape areas should be limited strictly to that necessary for plant growth. Excessive irrigation could result in saturation, weakening and possible swelling of the foundation soils.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

33. Our client should recognize that this report is prepared for the exclusive use of this project. Our professional services, findings, and recommendations were prepared in accordance with generally accepted engineering principles and practices. No other warranty, expressed or implied, is made.

34. The conclusions and recommendations contained in this report will not be considered valid after a period of two years unless the changes are reviewed, and the conclusions of this report are modified or verified in writing.

35. This report is issued with the understanding that it is the responsibility of the owner or his representative, to ensure the information and recommendations contained in this report are brought to the attention of the architect, engineer, and contractor. In all cases, the contractor shall retain responsibility for the quality of the work and for repairing defects regardless of when they are found. It is also the responsibility of the contractor for conforming to the project plans and specifications.

36. Our client should recognize that every effort made to evaluate the subsurface conditions at this site is based on the samples recovered from the test borings and the results of laboratory tests on these samples. The conclusions reached in this report were based on the conditions at the test boring locations. The owner or his representative should be reminded that unanticipated subsurface conditions are commonly encountered and cannot be fully determined by taking subsurface samples, and frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate these required extra costs.

Should you have any questions relating to the contents of this report, please contact our office at your convenience.

Very truly yours,

Wayne L. Ting, C.E.

Principal Engineer

Copies: 4 to Mr. Gill

Project No. 1855
7 May 2013

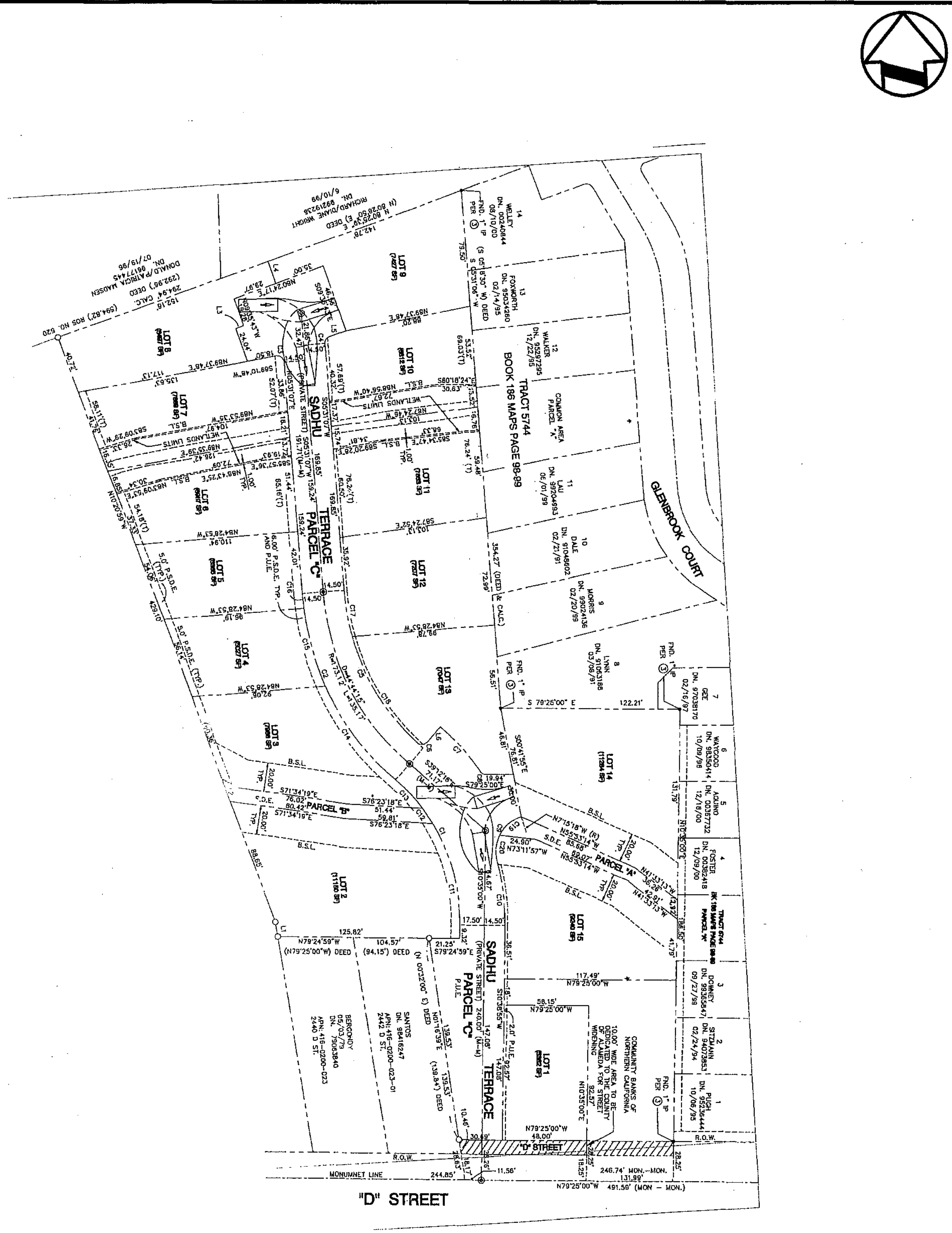
APPENDIX A

Tentative Map, Figure 1

Fill Slope, Figure 2.

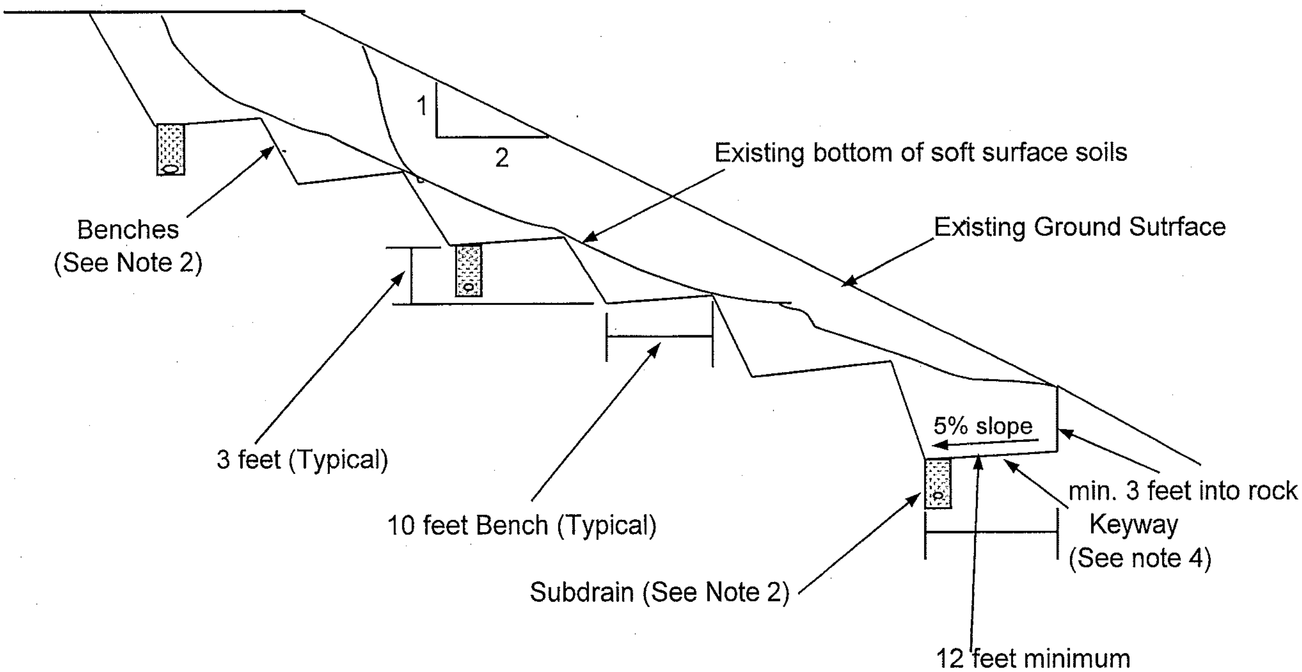
Drainage Behind Wall, Figure 3.

Foundation Drain Detail, Figure 4

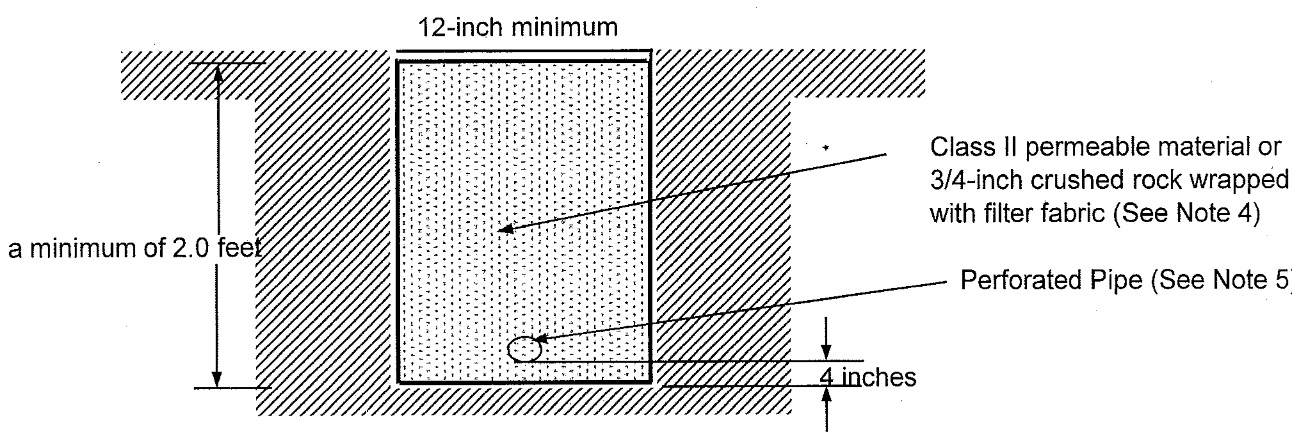


WAYNE TING & ASSOCIATES, INC.	**Site Plan**	Figure No. 1
GEOTECHNICAL CONSULTANTS	Scale: N/A	Page No. 12

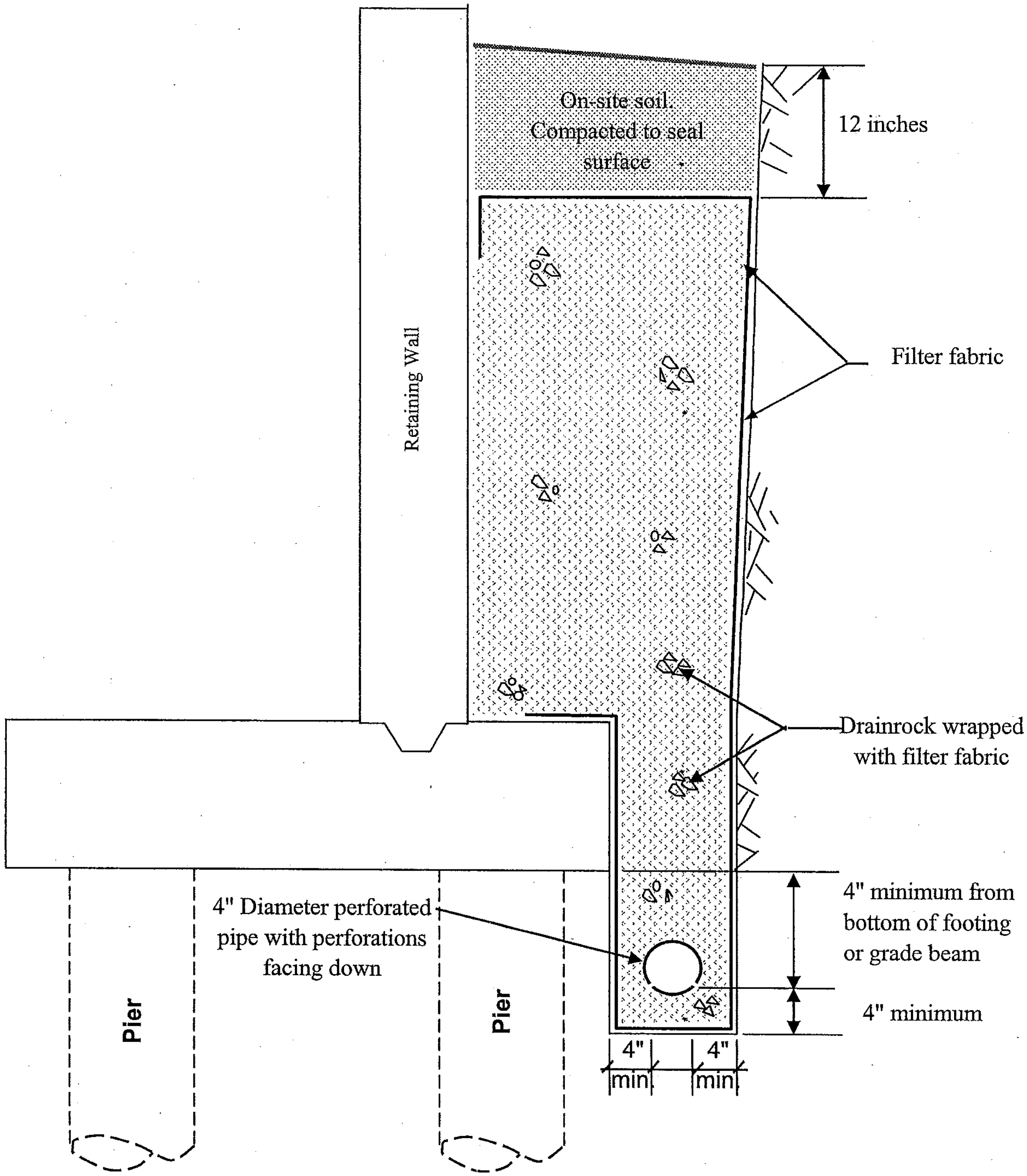
TYPICAL FILL SLOPE DETAILS



SUBDRAIN FOR KEYWAY

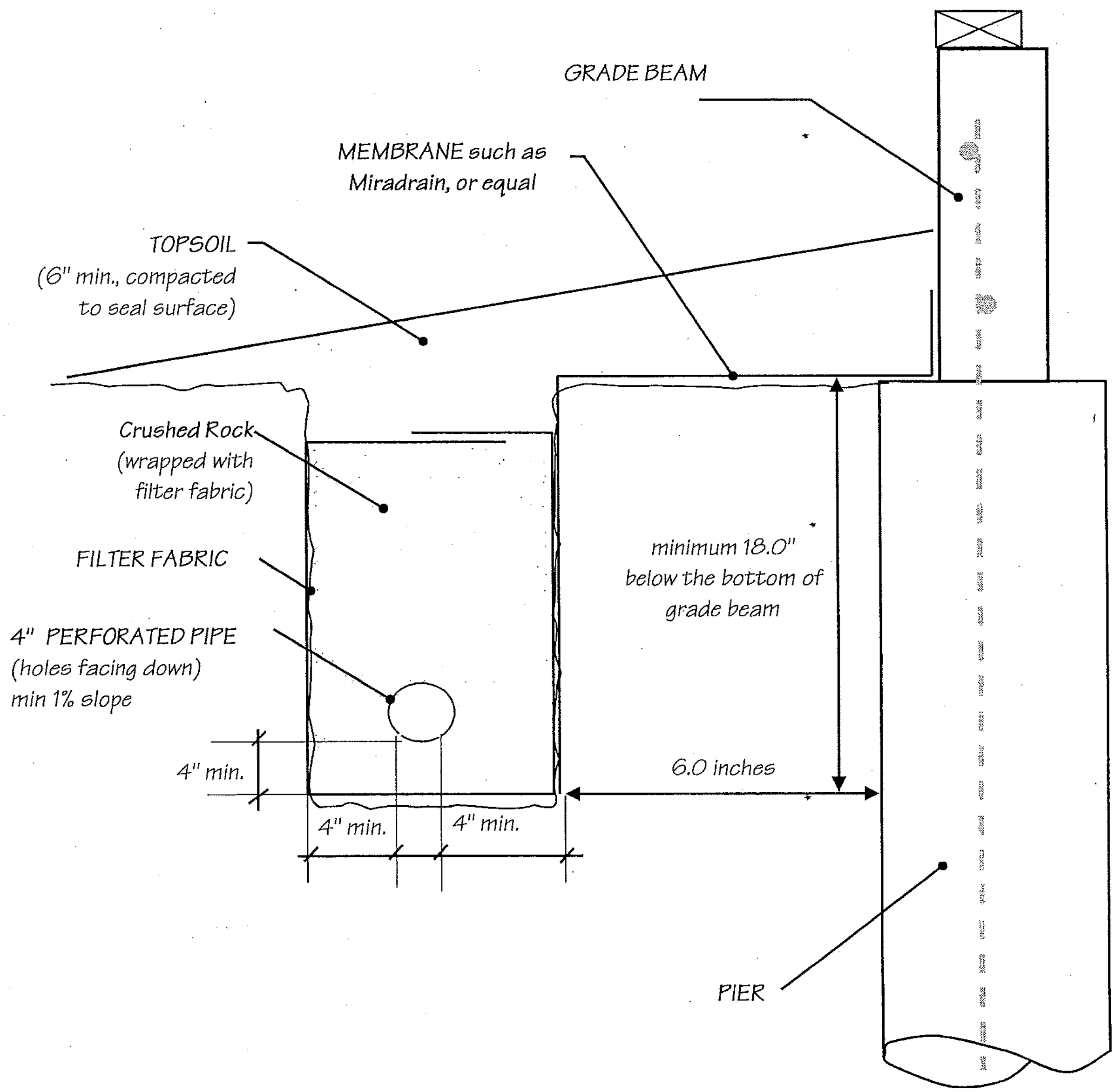


1. Where natural grade is steeper than 6:1, bench into bedrock as determine by WTAI.
2. Subdrain should discharge via a closed pipe to storm drain or suitable drainage area.
3. Keyway should extend at least 3 feet into bedrock as determined by the WTAI.
4. Class II permeable material given in Section 68-1.025, State of California Standard Specifications, July 1999 Edition.
5. Perforated pipe place perforations down, PVC pipe with a minimum diameter of four (4) inches, conforming to ASTM D-2751 SDR 35.



Note: Bottom of the subdrain trench and pipe should be sloped at least 1.0 percent.

WAYNE TING & ASSOCIATES, INC.	*Drainage Behind Retaining Wall*	Figure 3.
	Scale: N/A	*Page No. 14*

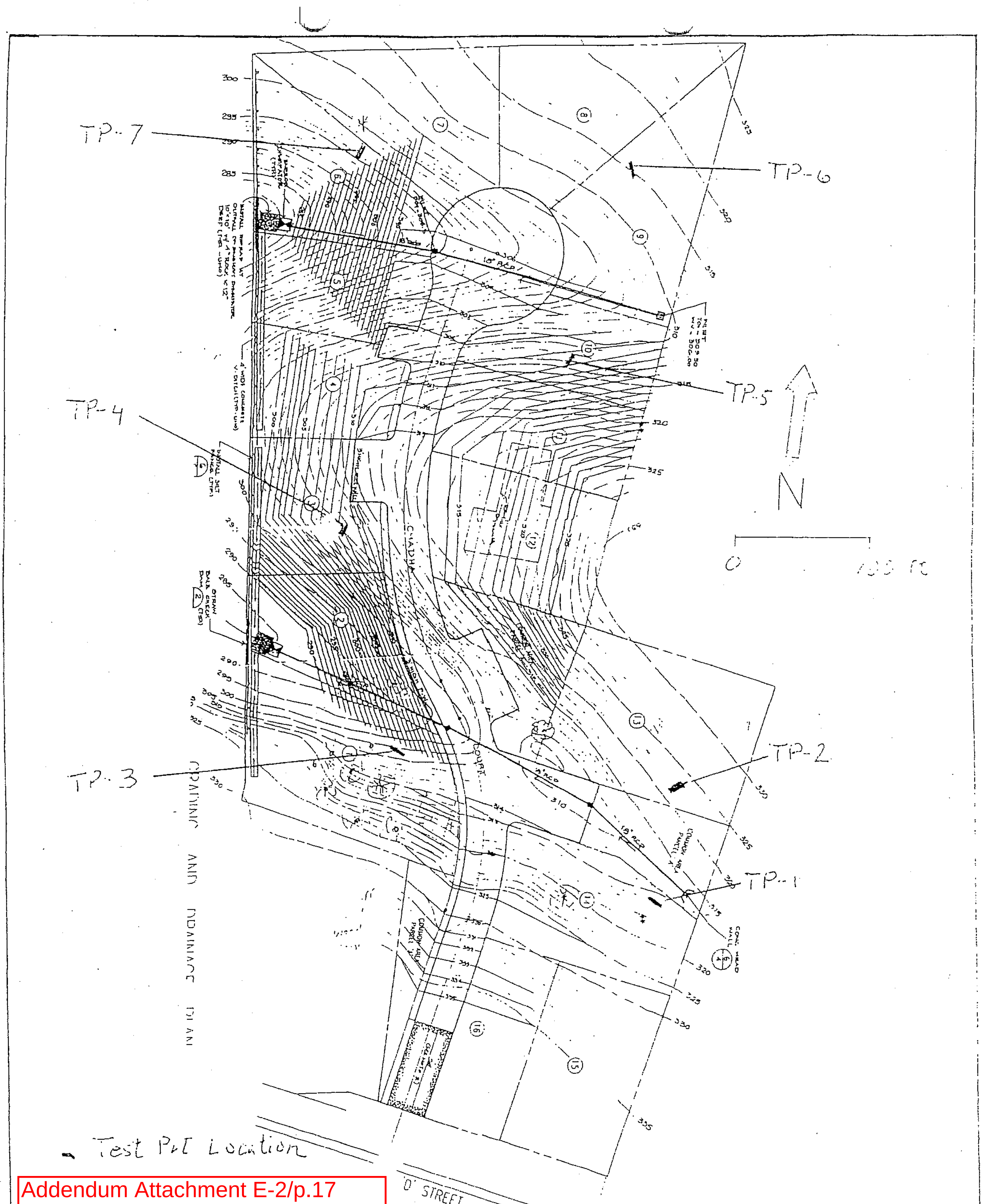


Addendum Attachment E-2/p.15

WAYNE TING & ASSOCIATES, INC.	FOUNDATION DRAIN	Figure 4
GEOTECHNICAL CONSULTANTS	Scale: N/A	Page No. 15

APPENDIX B

Site Plan, Boring Logs and Test Pits in References 2 and 4.



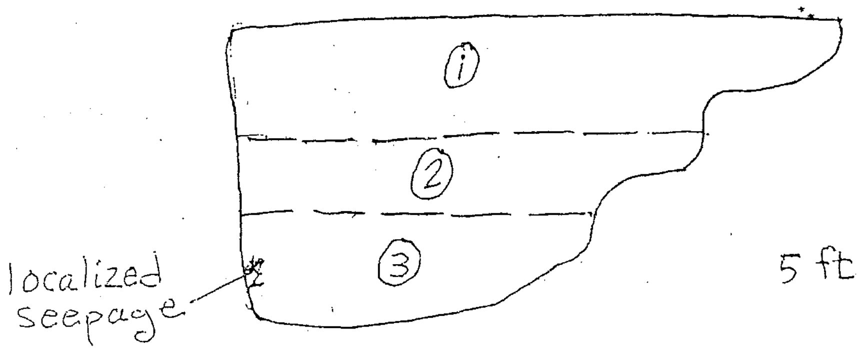
Addendum Attachment E-2/p.17

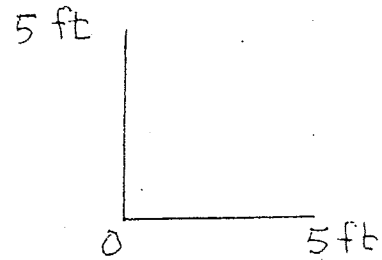
From Grading Plan Provided by Client

Buckley Engineering Associates	Job No. 02505.1 Approved Date 8-21-02	SITE PLAN D Street Hayward CA	Plate 3

TP-1

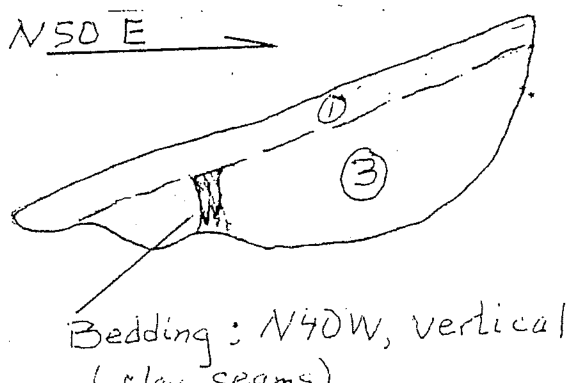
N 75 W →





TP-2

N 50 E →

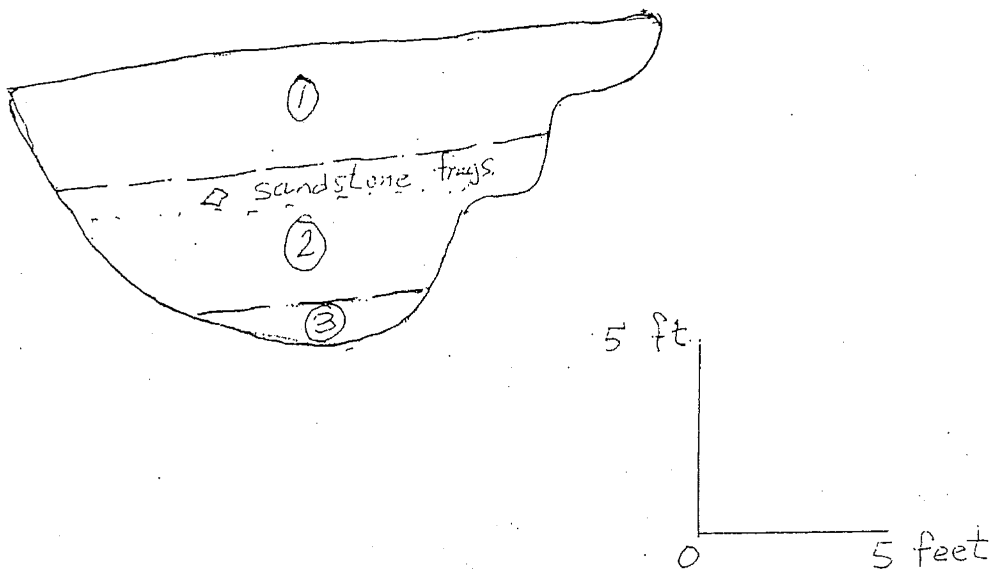


1. Gray-brown silty Clay
2. Brown silty clay, more clayey with rock fragments
3. Brown, fractured, massive, soft to hard, fine-grained weathered sandstone

Buckley Engineering Associates	Job No. 02505.1 Approved DWB Date 8-21-02	TEST PIT LOGS D Street Hayward, CA	Plate 4

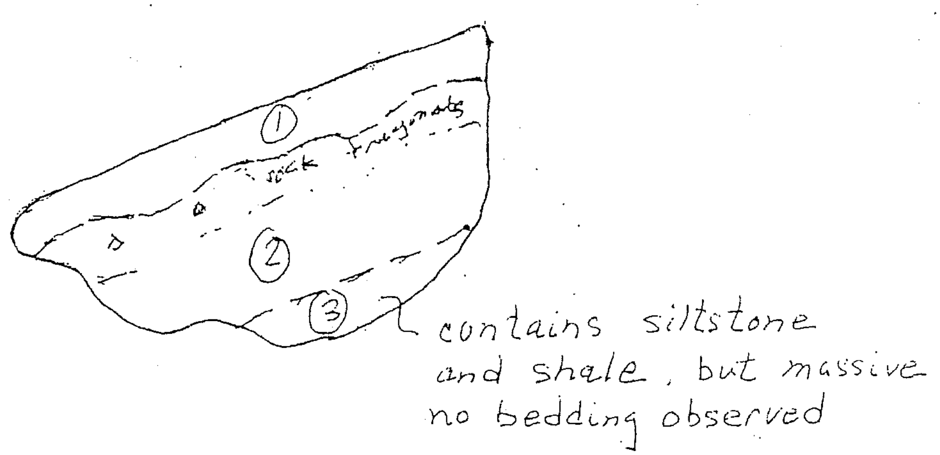
TP-3

N60W



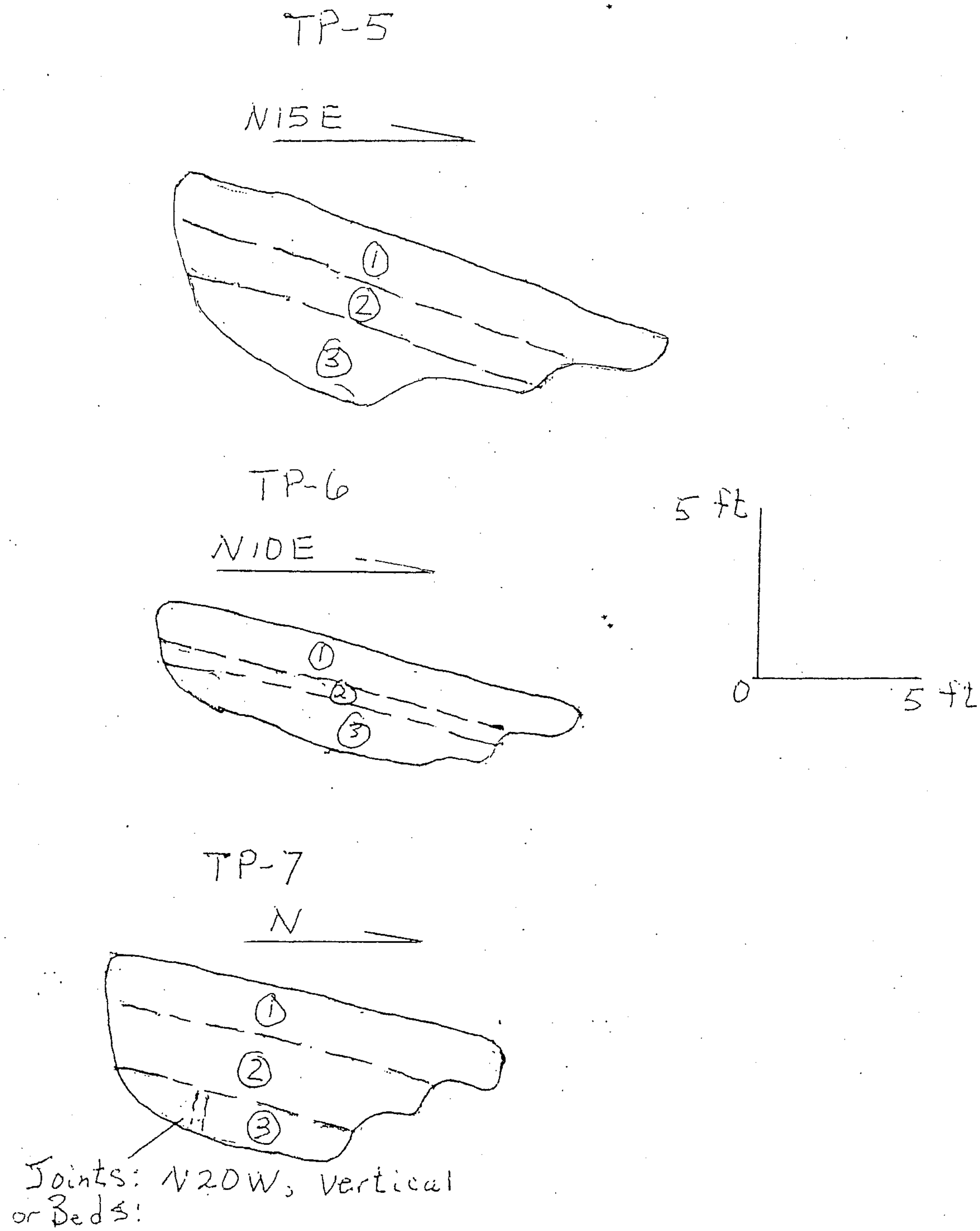
TP-4

N20E



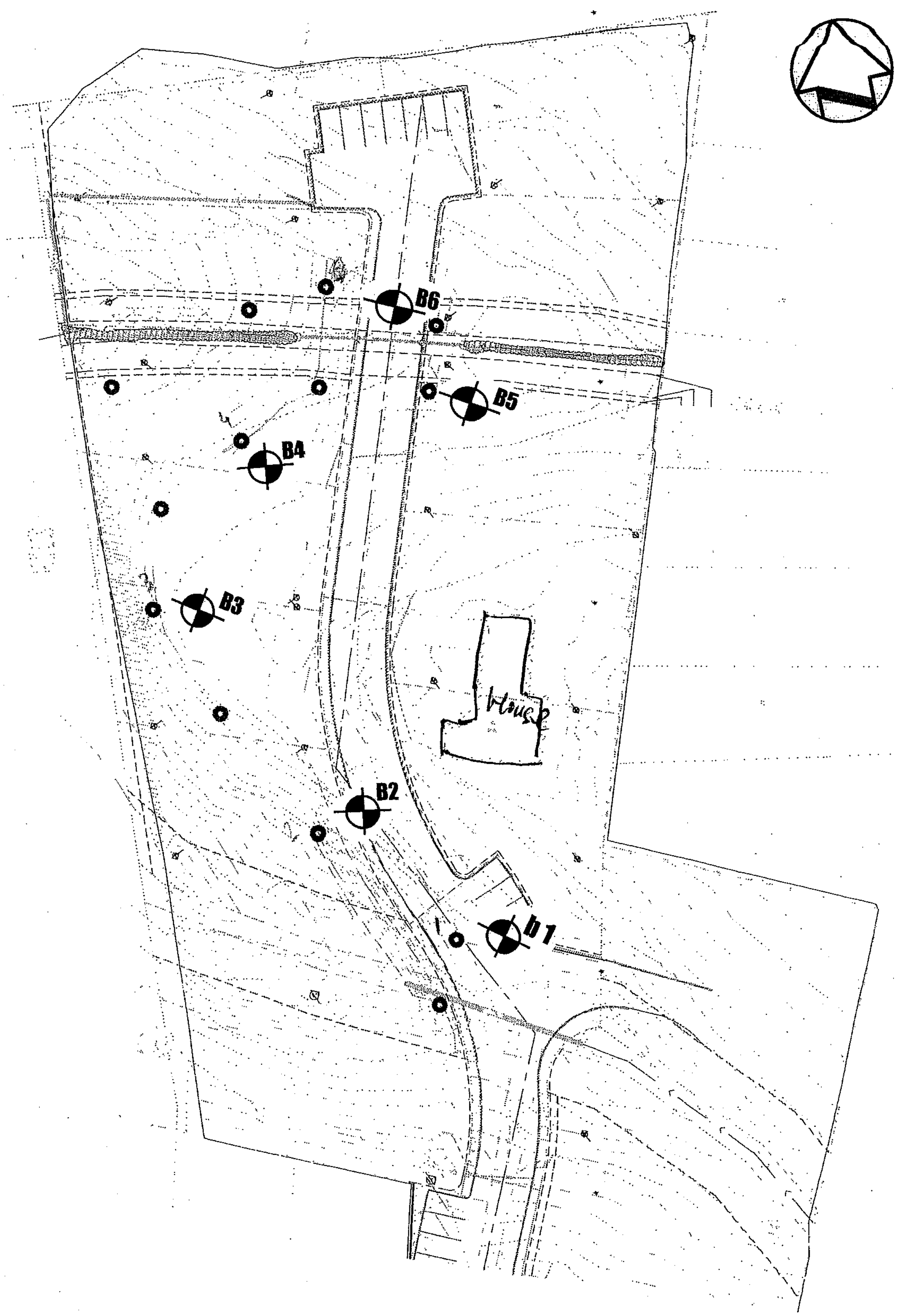
Addendum Attachment E-2/p.19

Buckley Engineering Associates	Job No. 02505.1 Approved DWB Date 8-21-02	TEST PIT LOGS D Street Hayward, CA	Plate 5



Buckley Engineering Associates	Job No. 02505.1 Approved DWB Date 8-21-02	TEST PIT LOGS D Street Hayward, CA	Plate 6

2492 D Street, Hayward, California | Project No. 1855 | 5 August 2010



WAYNE TING & ASSOCIATES, INC.	Site Plan	*Figure No. 1*
GEOTECHNICAL CONSULTANTS	Scale: N/A	*Page No. 1*

Addendum Attachment E-2/p.21

2492 D Street, Hayward, California | Project No. 1855 | 5 August 2010

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
	Medium brown silty sand and gravels mixture		SM					
1	with black ashpltic concrete debris							
2								
3		1-1		2		12.4		
4								
5								
6								
7	brown clayey sand and gravel	1-2		5		14.5		
8								
9								
10								
11								
12								
13		1-3		18		12.2		
14	with brick							
15	(bottom of uncontrolled fills)							
	Light brwon clayey sand, firm and moist		SC					
16								
17								
18								
19								
20	dnese							
21		1-4		>50		9.0		
22	Brown sandstone and minor water	1-5		>50				
	Boring terminated at 22 feet							
23	No groundwater encountered							
24								
25								

WAYNE TING & ASSOCIATES, INC.	*BORING LOG NO. 1*	*Figure No. 2*
GEOTECHNICAL CONSULTANTS	*Date Drilled:* 28 July 2010 *By:* TN	*Page No. 2*

2492 D Street, Hayward, California | Project No. 1855 | 5 August 2010

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
0	Brown clayey sand with gravel		SC					
7	(bottom of uncontrolled fills)							
7	Light brown sitly sand, medium dense and slightly moist		SM					
10.5	Brown sandstone	2-1		>50		11.3		
12		2-2		>50		12.6		
13	Boring terminated at 13 feet No groundwater encountered							

Depth scale: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25

WAYNE TING & ASSOCIATES, INC.

GEOTECHNICAL CONSULTANTS

BORING LOG NO. 2

Date Drilled: 28 July 2010 *By:* TN

Figure No. 3

Page No. 3

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
	Brown clayey sand and silty sand with gravel		SC-SM					
1								
2								
3	(bottom of uncontrolled fills)	3-1		8				
4	Light brown sitly sand, slightly moist and medium dense		SM					
5								
6	Brown sandstone							
7								
8		3-2		>50				
9	Boring terminated at 9.5 feet No groundwater encountered							
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								

WAYNE TING & ASSOCIATES, INC.	***BORING LOG NO. 3***	*Figure No. 4*
GEOTECHNICAL CONSULTANTS	*Date Drilled:* 28 July 2010 *By:* TN	*Page No. 4*

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
	Brown clayey sand and gravel, loose to medium dense, moist		SC					
1								
2								
3								
4								
5								
6		4-1		12		13.5		
7								
8								
9	(bottom of uncontrolled fills)							
10	Brown siltstone	4-2		25		15.8		
11	Boring terminated at 11 feet No groundwater encountered							
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								

WAYNE TING & ASSOCIATES, INC.	***BORING LOG NO. 4***	*Figure No. 5*
GEOTECHNICAL CONSULTANTS	*Date Drilled:* 28 July 2010 *By:* TN	*Page No. 5*

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
0–1	Brown clayey sand andsilty sand with gravel, firm and moist		SC-SM					
7	with grass							
10	with grass (bottom of uncontrolled fills)							
11	Brown silty sand with clay, medium dense and very moist	5-1	SM	8		22.4		
12.5	Brown sandstone							
16		5-2		>50				
16.5	Boring terminated at 16.5 feet. No groundwater encountered							

WAYNE TING & ASSOCIATES, INC.	***BORING LOG NO. 5***	*Figure No. 6*
GEOTECHNICAL CONSULTANTS	*Date Drilled:* 28 July 2010 *By:* TN	*Page No. 6*

2492 D Street, Hayward, California   Project No. 1855   5 August 2010

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
	Medium brown clayey sand with gravel, loose to medium dense, moist		SC					
1								
2								
3								
4								
5								
6	Medium brown clayey sand and silty sand with black asphaltic concrete, wood chips		SC					
7								
8		6-1		29		8.2		
9								
10								
11	(bottom of uncontrolled fills)							
12	Brown silty sand, medium dense and moist		SM					
13								
14	Brown sandstone							
15								
16		6-2		>50		15.6		
17	Boring terminated at 16.5 feet No groundwater encountered							
18								
19								
20								
21								
22								
23								
24								
25								

WAYNE TING & ASSOCIATES, INC.	***BORING LOG NO. 6***	*Figure No. 7*
GEOTECHNICAL CONSULTANTS	*Date Drilled:* 28 July 2010 *By:* TN	*Page No. 7*

Addendum Attachment E-2/p.27

Project No. 1855
5 August 2010

WAYNE TING & ASSOCIATES, INC.

GEOTECHNICAL CONSULTANTS
42329 Osgood Road, Unit A, Fremont, CA 94539
Phone (510) 623-7768 Fax (510) 623-7861

Mr. Ron Esau
R.V. Esau Development Company, Inc.
3620 Oakes Drive
Hayward, CA 94542

Subject: **UNCONTROLLED FILL INVESTIGATION**
Proposed Single-Family Subdivision
Tract No. 8022
2492 D Street
Alameda County, California

Reference: 1) Update of Geotechnical Investigation
And Supplemental Recommendations
By Wayne Ting and Associates, Inc.
Dated 8 December 2004

Dear Mr. Esau:

In accordance with your authorization, **Wayne Ting & Associates, Inc. (WTAI)** has completed an investigation for the existing uncontrolled fills at the subject site.

WTAI conducted the field investigation on 28 July 2010. The field investigation consisted of a site reconnaissance by the Project Engineer and excavation of six exploratory borings to maximum depths of 22.0 feet below the existing ground surface. The borings were excavated using a truck mounted drill-rig with a 4.5-inch stem-auger and a minuteman drill rig. The locations of the drilled borings are shown on Figure 1, Site Plan.

Soils encountered during the excavation operations were continuously logged in the field. Relatively undisturbed samples were obtained by dynamically driving 18 inches using a 3.0-inch outside diameter Modified California Sampler with a 140-pound hammer free falling 30 inches for truck. Blow counts were recorded for every 6-inch penetration interval, and reported corresponding to the last 12 inches of penetration. Blow count numbers presented in the boring log is converted to standard penetration blow count numbers. These samples were then sealed and returned to the laboratory for moisture testing. The classifications, descriptions, natural moisture contents, dry densities and depths from which the samples were obtained, are shown in the Boring Logs, Figure 2 through 7.

SUBSURFACE SOIL CONDITIONS

The subsurface soils in our drill borings consisted of 4 to 15 feet of medium brown to brown clayey sand and silty sand (uncontrolled fills), loose to medium dense and moist. Below the fills, brown silty sand and sandstone were encountered to the maximum depth explored of 22.0 feet.

No groundwater was encountered at the time of the field study. It is noted that fluctuations in the groundwater table are anticipated to vary with respect to seasonal rainfall.

RECOMMENDATIONS

According to the above-mentioned test borings, approximately 4 to 15 feet of uncontrolled fills were encountered on the slope of the subject site. These fills are loose and will a high possibility to slide downhill or to creek. To avoid the potential soil sliding, these fills must be immediately removed to the native soils.

Erosion control should then be installed before the raining season. WTAI should observe the grading operations and erosion control measurements.

Should you have any questions relating to the contents of this report, please contact our office at your convenience.

Very truly yours,

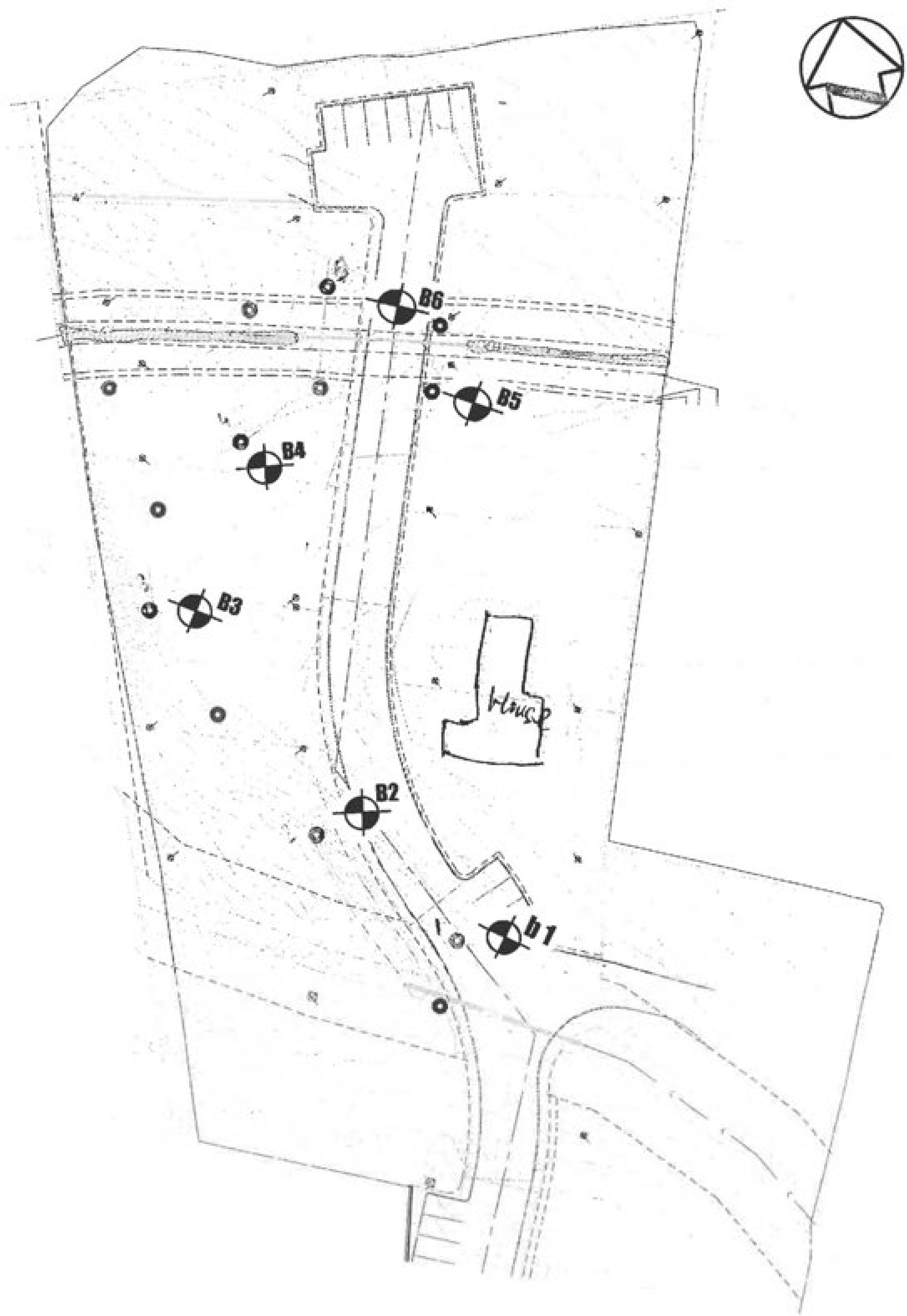
WAYNE TING & ASSOCIATES, INC.

Wayne Ting

Wayne L. Ting, C.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
WAYNE L. TING
No. C 46276
Exp: 12/31/10
CIVIL
STATE OF CALIFORNIA

Copies: 3 to Mr. Esau



WAYNE TING & ASSOCIATES, INC.	Site Plan	*Figure No. 1*
GEOTECHNICAL CONSULTANTS	Scale: N/A	*Page No. 1*

2492 D Street, Hayward, California | Project No. 1855 | 5 August 2010

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
	Medium brown silty sand and gravels mixture with black ashpltic concrete debris		SM					
1								
2								
3		1-1		2		12.4		
4								
5								
6								
7	brown clayey sand and gravel	1-2		5		14.5		
8								
9								
10								
11								
12								
13		1-3		18		12.2		
14	with brick (bottom of uncontrolled fills)							
15	Light brwon clayey sand, firm and moist		SC					
16								
17								
18								
19								
20	dnese							
21		1-4		>50		9.0		
22	Brown sandstone and minor water	1-5		>50				
	Boring terminated at 22 feet No groundwater encountered							
23								
24								
25								

WAYNE TING & ASSOCIATES, INC.	***BORING LOG NO. 1***	*Figure No. 2*
GEOTECHNICAL CONSULTANTS	*Date Drilled:* 28 July 2010 *By:* TN	*Page No. 2*

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
0	Brown clayey sand with gravel		SC					
1								
2								
3								
4								
5								
6								
7	(bottom of uncontrolled fills) Light brown sitly sand, medium dense and slightly moist		SM					
8								
9								
10								
11	Brown sandstone	2-1		>50		11.3		
12		2-2		>50		12.6		
13	Boring terminated at 13 feet No groundwater encountered							
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								

WAYNE TING & ASSOCIATES, INC.	***BORING LOG NO. 2***	*Figure No. 3*
GEOTECHNICAL CONSULTANTS	*Date Drilled:* 28 July 2010 *By:* TN	*Page No. 3*

2492 D Street, Hayward, California — Project No. 1855 — 5 August 2010

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
	Brown clayey sand and silty sand with gravel		SC-SM					
1								
2								
3								
	(bottom of uncontrolled fills)	3-1		8				
4	Light brown silty sand, slightly moist and medium dense		SM					
5								
6	Brown sandstone							
7								
8		3-2		>50				
9	Boring terminated at 9.5 feet No groundwater encountered							
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								

WAYNE TING & ASSOCIATES, INC.	***BORING LOG NO. 3***	*Figure No. 4*
GEOTECHNICAL CONSULTANTS	*Date Drilled:* 28 July 2010 *By:* TN	*Page No. 4*

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
0–9	Brown clayey sand and gravel, loose to medium dense, moist		SC					
6		4-1		12		13.5		
9	(bottom of uncontrolled fills)							
10	Brown siltstone	4-2		25		15.8		
11	Boring terminated at 11 feet No groundwater encountered							

Depth scale: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25

WAYNE TING & ASSOCIATES, INC.	***BORING LOG NO. 4***	*Figure No. 5*
GEOTECHNICAL CONSULTANTS	*Date Drilled:* 28 July 2010 *By:* TN	*Page No. 5*

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
0	Brown clayey sand andsilty sand with gravel, firm and moist		SC-SM					
7	with grass							
10	with grass (bottom of uncontrolled fills)	5-1		8		22.4		
11	Brown silty sand with clay, medium dense and very moist		SM					
12.5	Brown sandstone							
16		5-2		>50				
16.5	Boring terminated at 16.5 feet No groundwater encountered							

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry Density)	Pocket Penet. (T.S.F)	Remarks
	Medium brown clayey sand with gravel, loose to medium dense, moist		SC					
1								
2								
3								
4								
5								
6	Medium brown clayey sand and silty sand with black asphaltic concrete, wood chips		SC					
7								
8		6-1		29		8.2		
9								
10								
11	(bottom of uncontrolled fills)							
12	Brown silty sand, medium dense and moist		SM					
13								
14	Brown sandstone							
15								
16		6-2		>50		15.6		
17	Boring terminated at 16.5 feet No groundwater encountered							
18								
19								
20								
21								
22								
23								
24								
25								

WAYNE TING & ASSOCIATES, INC.	***BORING LOG NO. 6***	*Figure No. 7*
GEOTECHNICAL CONSULTANTS	*Date Drilled:* 28 July 2010 *By:* TN	*Page No. 7*

WAYNE TING & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

Project No. 1855
1 April 2010

Mr. Ron Esau
R.V. Esau Development Company, Inc.
3620 Oakes Drive
Hayward, CA. 94542

Subject: **UPDATE OF GEOTECHNICAL INVESTIGATION AND SUPPLEMENTAL RECOMMENDATIONS**
Proposed Single-Family Subdivision
Tract No. 8022
2492 D Street
Alameda County, California

References:
1) Geotechnical Investigation
By Cleary Consultants, Inc.
Dated 7 July 1989
2) Geologic Investigation
By Buckley Engineering Associates, Dated 21 August 2002
3) Geologic Report Update
By Buckley Engineering Associates, Dated 19 September 2005

Dear Mr. Esau:

At your request, Wayne Ting & Associates, Inc. (WTAI) performed a reconnaissance of the subject site and reviewed the referenced materials to determine if the geotechnical recommendations provided in References 1 may apply to construction of the proposed development at the subject site.

Based on our reconnaissance and review, it is the opinion of WTAI that the referenced reports (References 1 and 2) present acceptable data and geotechnical recommendations for the design and construction of the subject project. However, the supplemental recommendations provided below should be incorporated into the project design.

GRADING

Cut and fill transition at garage concrete slabs-on-grade area may experience abrupt differential settlement causing significant distress. This condition can be mitigated by scarifying the cut portion of the transition garage pad a minimum depth of 12 inches. The scarified material should be properly moisture-conditioned to at least 2 percent above optimum moisture content and be

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recompacted to a minimum relative compaction of 90 percent. It is noted that a minimum three feet of uniform engineered fill should be constructed under the entire garage area. The fill should be placed in thin lifts not exceeding 8 inches in uncompacted thickness and compacted to the abovementioned compaction requirements.

Uncontrolled fills may be encountered on the proposed lots 3 through 5. The locations and depths of these fills should be determined by WTAI during the grading operations. All uncontrolled fills should be overexcavated and replaced with engineered fills.

SLOPES

In general, all fill slopes should not be steeper than 2:1 (horizontal:vertical). Cut slopes in stiff natural materials should not exceed 2:1 (H:V).

A shear key must be established at the toe of all fill slopes where the natural hill slope exceeds 6:1 (horizontal: vertical). The shear key must be at least 12 feet in width and cut 3 feet into the underlying rock. The bottom of the keyway excavation should be sloping back into the hillside at a minimum gradient of 5 percent. The location and depth of the keyway and subdrain should be determined by WTAI during grading operations. Subsequent benches should be placed at vertical heights of 3 feet and should extend horizontally into the rock. A typical section is presented in Figure 4, Fill Slope Detail.

During the grading operations, fill slopes must be compacted and should be over-constructed. At the completion of grading operations, the excess fill or loose soils existing on the slopes should be cut to a firm and adequately designed slope grade. Track-walking of the slope surface should only be utilized to seal the surface.

Before work is stopped due to heavy rains, a positive gradient away from slopes should be provided to carry surface runoff water away from the slope and to areas where erosion can be controlled. After the completion of slope grading, the exposed cut and fill slopes should be planted with deep-rooted native plants to minimize erosion. After grading is completed and WTAI has finished the observation of the work, no further grading shall be done unless it is approved by WTAI. Some minor erosion on slopes should be expected. Thus, periodic maintenance is required.

It is noted that the test pits mentioned in Reference 2 were loosely backfilled. Therefore, these pits should be overexcavated and backfilled with engineered fills and compacted to not less than 95 percent relative compaction.

UNIFORM BUILDING CODE SITE CHARACTERIZATION

The significant earthquakes which occur in the Bay Area are generally associated with crustal movements along well defined active fault zones. According to the published maps by International

Conference of Building Officials (I.C.B.O.), in February 1998, the nearest active fault to the subject site is the The significant earthquakes which occur in the Bay Area are generally associated with crustal movements along well defined active fault zones. According to the published maps by International Conference of Building Officials (I.C.B.O.), in February 1998, the nearest active fault to the subject site is the Hayward Fault which is located approximately 1.9 kilometers southwest. Therefore, the potential for surface fault trace rupture is considered to be negligible. We anticipate the proposed structure will subject to very strong ground shaking during the lifetime of the building structure.

Based on the geologic information and the distance to the seismic source, the Hayward fault is the controlling fault of the property. Therefore, according to chapter 16 of the California Building Code 2001 (CBC), the site seismic design values have been provided as follows:

CBC Category/Coefficient		Design Value
(Figure 16-2)	Seismic Zone	4
(Table 16-I)	Seismic Zone Factor	0.4
(Table 16-J)	Soil Profile Type	S_D
(Table 16-U)	Seismic Source Type	A
(Table 16-S)	Near Source Factor, Na	1.50
(Table 16-T)	Near Source Factor, Nv	2.00

The above-described acceleration and design values should only be considered reasonably best estimates. There can be significant deviations and variations from the indicated values due to various uncertainties, geologic factors and other specific conditions at the site.

FOUNDATIONS

Pier design criteria were provided in Page 11 of Reference 1. Pier should have a minimum diameter of 16 inches and 10 feet penetrating into rock.

RETAINING WALL

The following design parameter should be used for structural design of proposed retaining walls at the subject site. The drainage detail behind the wall is provided in Figure 5.

Slope Inclination Behind Wall (Horizontal : Vertical)	Equivalent Fluid Weight (Pounds Per Cubic Foot) Unrestrained	Restrained	Passive Resistance (p.s.f.)	Coefficient of Friction
Level	45	65	300	0.3
2:1	65	85	300	0.3

CONCRETE SLAB-ON-GRADE

Concrete slabs should not be doweled into the foundation perimeter and should be reinforced using at least No. 4 bars at 18-inch on centers to reduce cracking.

DRAINAGE

A foundation drain system should be constructed around the perimeter foundations. The foundation drain should be constructed at a lateral distance of 6.0 inches from the foundation and extended a minimum depth of 18 inches below the bottom of the grade beam. The recommended subdrain detail is presented in Figure 6. The perforated pipe shown in Figure 6 will pass into a solid line pipe at the end drain then be directed to a suitable discharge area. Cleanout risers should be provided at the upgradient end of the perforated pipe, at sharp bends, and at 100 foot maximum intervals.

PRELIMINARY PAVEMENT SECTION

The top 10 inches of street subgrade should be scarified and recompacted to a minimum relative compaction of 95% and at 2% above the optimum moisture content as determined by ASTM D1557-91 Laboratory Test Procedure.

Aggregate subbase should then be placed on top of the subgrade and compacted to a minimum relative compaction of 95%. Class II aggregate base must also be compacted to 95% relative compaction. The class II aggregate base should conform to the requirements of Standard Specifications of Caltrans, Section 26-1.02A.

Pavement Sections: The following recommended pavement sections are based on Traffic Indices (T.I.) of 4, 5 and 6, and assuming R-value of 15.

Traffic Index	Asphaltic Concrete	Class II Aggregate	Aggregate Subbase
4	3.0"	7.0"	10.0"
5	3.0"	10.0"	13.0"
6	4.0"	11.0"	15.0"

EARTHQUAKE-INDUCED LANDSLIDE ANALYSIS

Background

It is noted that the proposed lot 2, is located within the earthquake-induced landslide zones based on the California Seismic Hazard Zones, Hayward Quadrangle map, dated July 2, 2003, the proposed development will need to address the potential of permanent ground displacement during earthquakes. Our evaluation is based on California Department of Conservation, Division of Mines and Geology 's Special Publication 117 (SP 117), Guidelines for Evaluation and Mitigating Seismic Hazards in California. We conducted seismic slope stability analysis that is consistent with the "Recommended Procedure for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California," developed by the ASCE Implementation Committee, chaired by Thomas F. Blake, dated June 2002 (Blake et al 2002). The results of analysis are presented in Appendix B.

Site Description

It is noted that a gully was observed in the middle of the proposed lot 2. The ground surface at the south part of lot 2 with inclinations 2:1 (horizontal:vertical) to 3:1 slopes downward from south to north. The south lot consists of trees and bushes. The ground surface at the north part of lot 2 with inclinations 2:1 (horizontal:vertical) to 3:1 slopes downward from north to south. The north lot was covered by dense trees and bushes.

Supplemental Investigation

In order to perform slope stability analysis, WTAI performed additional field investigation which was conducted on August 24, 2005 and consisted of a site reconnaissance by the project engineer and the excavation of two exploratory borings using a minuteman drilling rig with a 3.0-inch auger. The approximate locations of the borings are shown on the Site Plan, Figure 1.

The soils encountered during the excavation operations were continuously logged in the field. Relatively undisturbed samples were obtained by dynamically driving a 2.5 inch outside diameter Modified California sampler with a 140 pound hammer falling 30 inches. Samples were then sealed and returned to our laboratory for testing. The testing results are shown on the boring logs, Figures 2 and 3 of Appendix A.

LABORATORY TESTS

Classification

The field classifications of the samples were visually verified in the laboratory in accordance with the Unified Soil Classification System. These classifications are presented on the Boring Log, Figures 2 and 3.

Moisture and Density

The natural moisture contents and/or dry weights were determined for selected samples obtained during our field investigation. This data is presented in the aforementioned boring logs.

Direct Shear

Direct shear tests performed by Cooper Laboratory Testing were performed on several samples to determine the strength parameters of the soils. The laboratory testing results are shown in the Boring Logs and Figures 4 and 5.

Subsurface Soil Conditions

The following soil descriptions were derived from our site reconnaissance and the information obtained from our exploratory boring samples. Detailed descriptions of the materials encountered in the exploratory boring and the results of the laboratory testings are presented on the Boring Logs.

Boring 1, encountered 7.0 feet of medium brown to brown, stiff and moist, sandy silt, followed by moist and hard, brown silty clay. Below the clay, medium dense and moist, fractured and weathered sandstone was encountered to the maximum depth explored of 18.5 feet.

Boring 2, encountered 2.5 feet of brown, stiff and moist, sandy clay, followed by medium dense and moist, fractured and weathered sandstone to the maximum depth explored of 7.5 feet.

Groundwater was encountered at 9.0 feet below the ground surface in the exploratory boring 1 at the time of the field study. Ground water was not encountered in boring 2. However, fluctuations in the groundwater table are anticipated to vary with seasonal rainfall variations.

Soil and Rock Geotechnical Parameters

The laboratory test results, our field observations and engineering experience form the basis for using the following engineering properties in our stability analysis:

Material	Unit Weight (p.c.f.)	Cohesion (p.s.f.)	Friction Angle (degrees)
Silty clay (native)	125	540	16.0
Sandstone	130	1,000	35

The computer program XSTABL, Version 5, developed by Dr. Sunil Sharma was used to calculate factors of safety for the native slopes. A representative slope profile, Section A-A was selected for the analysis. The stability analysis was performed using undrained strength parameters under seismic condition.

Seismic Coefficient

The seismic coefficient used for the screening analysis (Blake and others, 2002), was estimated as a corrected mean horizontal acceleration on "soft rock" representing the 475-year return period (10% in 50 year hazard level) shown in the following equation:

where k = seismic coefficient for pseudo-static stability analysis

f_{eq} = site seismicity factor

MHA_r = maximum horizontal acceleration at the site for a soft rock site condition

g = acceleration of gravity

Utilizing the California Geological Survey (CGS) resources for Probabilistic Seismic Hazards Mapping Ground Motion, we estimate the site MHA_r to be 0.712g. Based on USGS probabilistic seismic hazard de-aggregation analysis, the corresponding earthquake modal magnitude and distance for the site are 7.1 and 1.9 km, respectively. For a magnitude 7.1 earthquake at 1.9 km from the site, we estimate the f_{eq} to be 0.38. Therefore, the seismic coefficient, k, for the slope stability analysis was estimated to be 0.27 g. See Appendix A for detailed calculations.

Stability Analysis Results

The results of the stability analysis are summarized as follows:

Failure Plane	Loading Condition	Pseudo Static Factor of Safety
Circular	Undrained Strength	1.9

A factor of safety of 1.2 or greater for the pseudo-static analyses is considered to be adequate. The result of the pseudo-static factor of safety at the subject site is 1.9. Therefore, the analysis indicates the existing slopes meet the minimum factor of safety criteria stated in SP 117. It is our opinion that permanent ground deformation during strong earthquakes would be small, if any.

GENERAL CONSIDERATIONS

It is recommended that the WTAI be given the opportunity to review the grading and foundation plans and specifications when completed, to evaluate compliance with the recommendations provided in this report.

It is further recommended that WTAI be retained for testing and observation during all grading and foundation construction phases to help determine that the design requirements are fulfilled. WTAI should be notified at least 48 hours prior to grading and/or foundation operations on this project.

Any work related to the grading and/or foundation operations performed without the direct observation of WTAI will invalidate the recommendations of this report.

ALL OTHER RECOMMENDATIONS CONTAINED IN REFERENCE 1 THAT ARE NOT SPECIFICALLY MODIFIED HEREIN SHOULD BE STRICTLY FOLLOWED.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

Our professional services, findings, and recommendations were prepared in accordance with generally accepted engineering principles and practices. No other warranty, expressed or implied, is made.

At the present date, the findings of this report are valid for the property investigated. However, changes in the conditions of a property can occur with the passage of time. In the event that any changes in the nature, design, or location of the building are planned, the conclusions and recommendations contained in this report shall not be considered valid after a period of two (2) years, unless the changes are reviewed and conclusions of this report modified or verified in writing.

WTAI assumes full responsibility for the implementation of only the geotechnical recommendations provided in the referenced materials and this letter. WTAI will also be the geotechnical consultant of the record.

Should you have any questions or require additional information, please contact our office at your convenience.

Very truly yours,

WAYNE TING & ASSOCIATES, INC.

Wayne L. Ting, C.E.

Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
WAYNE L. TING
No. C 46276
Exp: 12/31/10
CIVIL
STATE OF CALIFORNIA

Copies: 5 to Mr. Esau

APPENDIX A

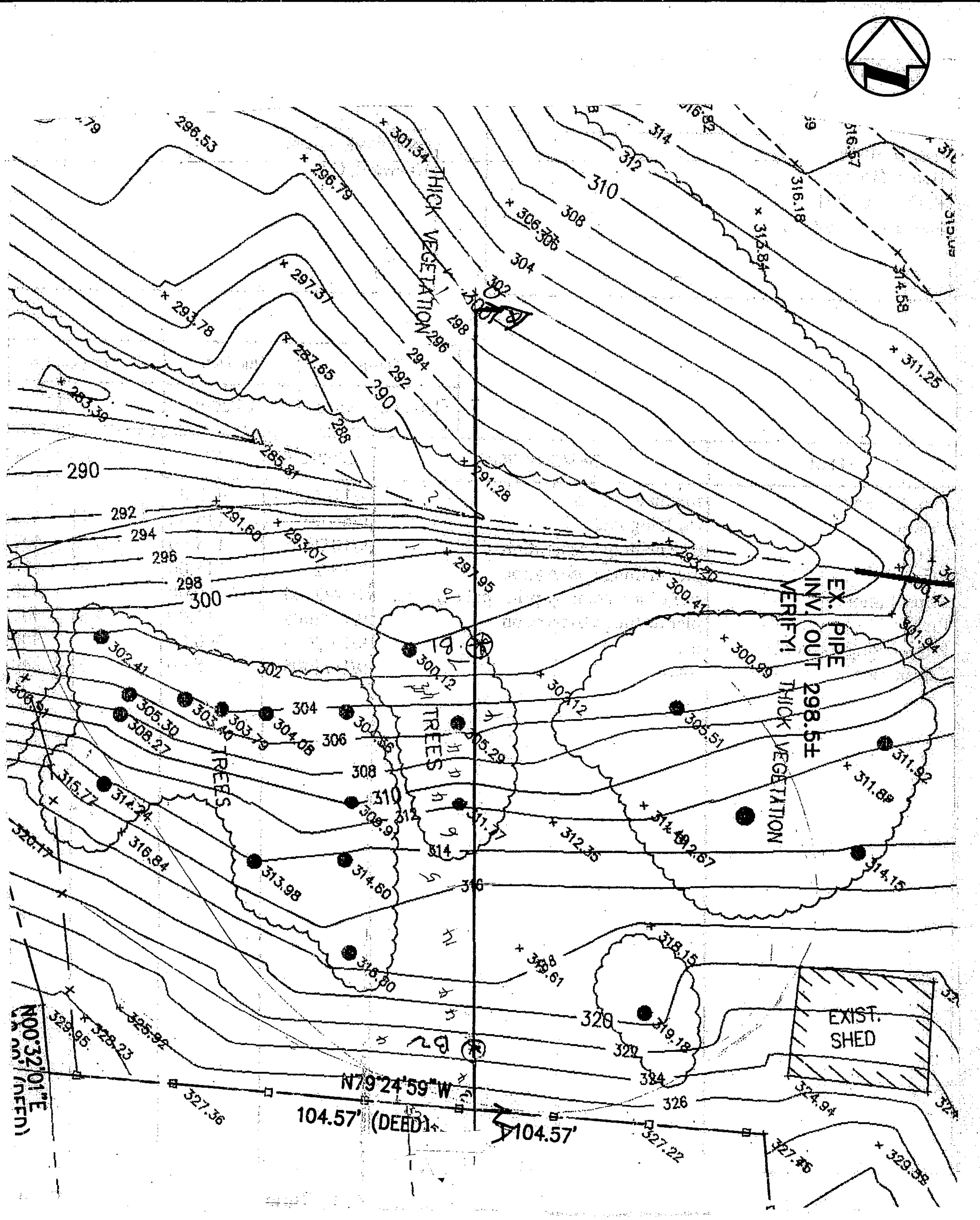
Site Plan, Figure 1

Boring Logs, Figures 2 and 3.

Fill Slope, Figure 4.

Drainage Behind Wall, Figure 5.

Foundation Drain Detail, Figure 6



WAYNE TING & ASSOCIATES, INC.	*SITE PLAN*	*Figure No. 1*
	Scale:1"=20'	*Page No. 10*

2492 D Street, Hayward, California | Project No. 1855 | 1 April 2010

Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry)	Pocket Penet. (T.S.F)	Remarks
	Medium brown sandy silt, moist and stiff		ML					
1	became to brown							
2								
3		1-1		13	-	16.9	-	
4	Yellowish brown							
5								
6								
7	Brown silty clay, hard and moist		CL					
8		1-2		>50	107.6	15.0	-	
9	water at 9 feet Brown fractured and weather sandstone, medium dense							
10								
11								
12		1-3		>50	-	-	-	
13	Boring terminated at 18.5 feet. Groundwater encountered at 9.0 feet							
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25	Addendum Attachment E-4/p.11							

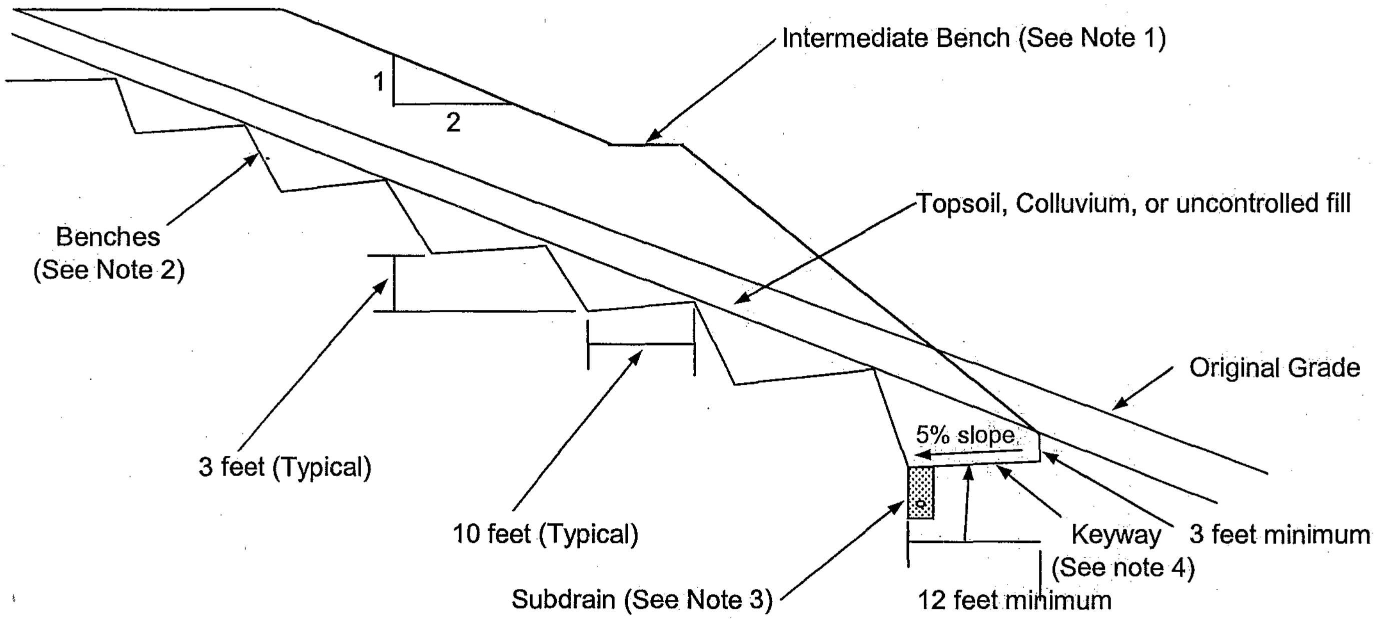
WAYNE TING & ASSOCIATES, INC.	*BORING LOG NO. 1*	*Figure No. 2*
GEOTECHNICAL CONSULTANTS	*Date Drilled:* 24 August 2005 *By:* R.W.	*Page No. 11*

2492 D Street, Hayward, California | Project No. 1855 | 1 April 2010

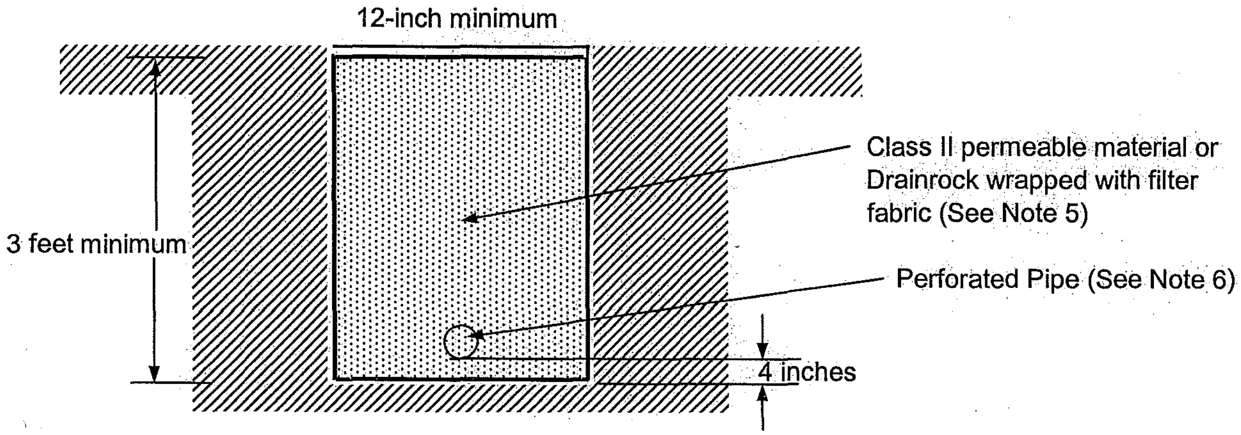
Depth (Feet)	Description	Sample No.	Unified Soil Classification	Blows/Foot (350 Ft.-Lbs)	Dry Density (P.C.F)	Moisture (% Dry	Pocket Penet. (T.S.F)	Remarks
	Medium brown sandy clay, moist and hard		CL					
1	became to brown							
2								
3	Yellowish brown freactured and weathered sandstone, moist and medium dense	2-1		>50	111.6	12.6	>4.5	
4								
5								
6	hard							
7		2-2		>50	-	11.8	-	
8	Boring terminated at 7.5 feet. No groundwater encountered							
9								
10								
11								
12								
13								
14								
15								
16								
17								
18								
19								
20								
21								
22								
23								
24								
25								

WAYNE TING & ASSOCIATES, INC.	*BORING LOG NO. 2*	*Figure No. 3*
GEOTECHNICAL CONSULTANTS	*Date Drilled:* 24 August 2005 *By:* R.W.	*Page No. 12*

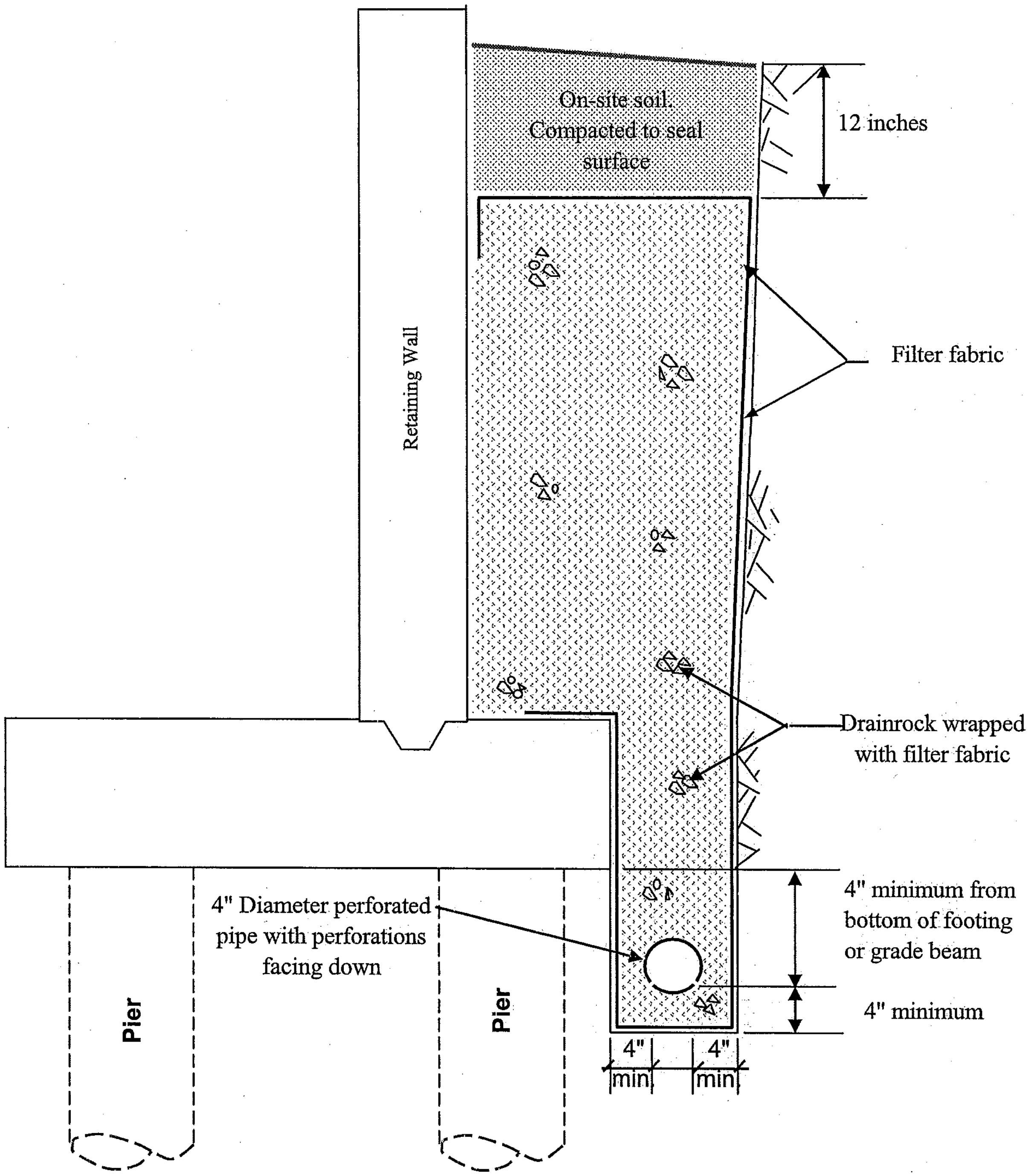
FILL SLOPE DETAILS



SUBDRAIN FOR KEYWAY

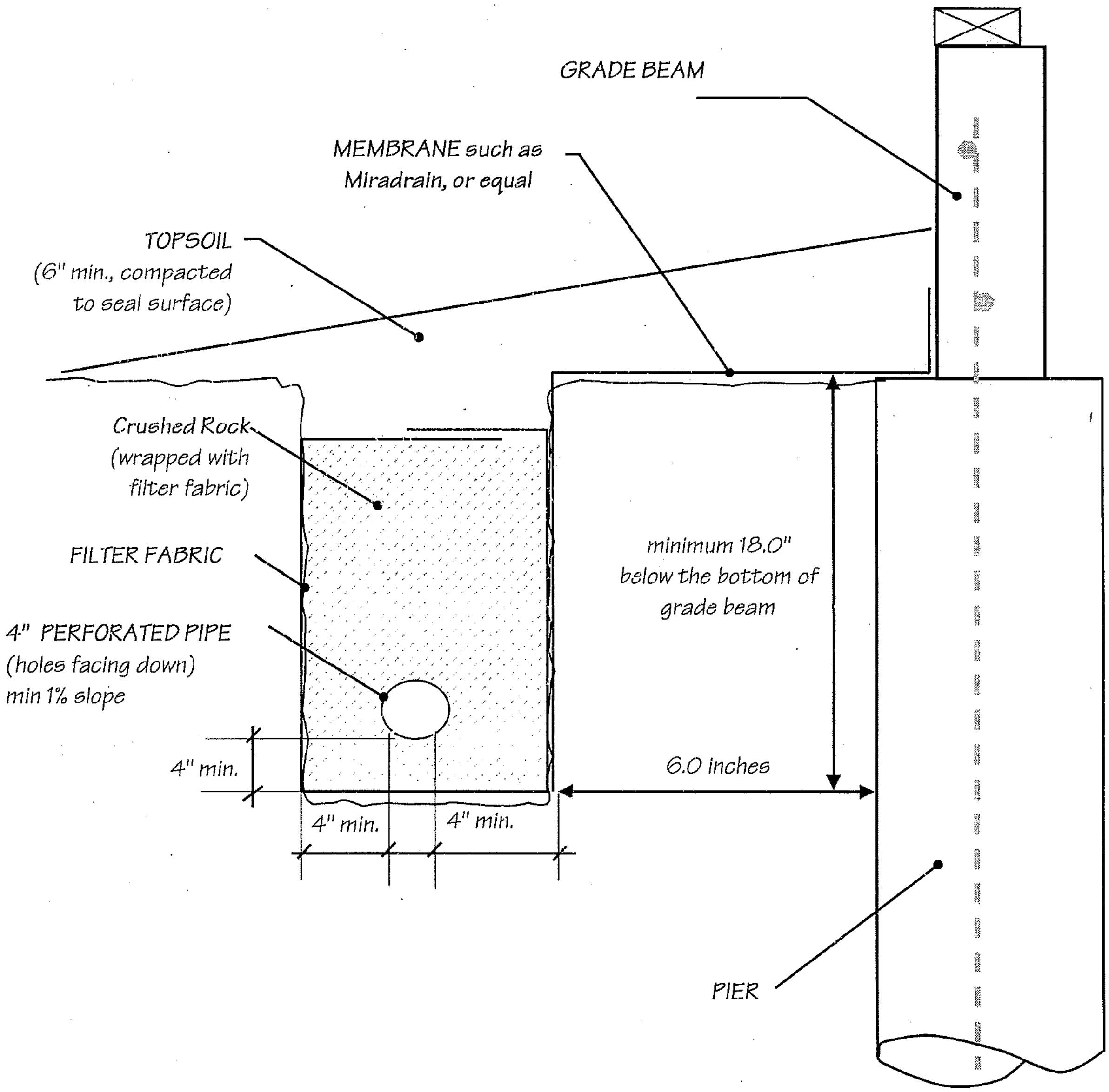


1. Intermediate benches should be spaced according to recommendations presented in this report.
2. Where natural grade is steeper than 6:1, bench into stiff soil or bedrock as determine by WTAI.
3. Subdrain should discharge via a closed pipe to storm drain or suitable drainage area.
4. Keyway should extend at least 3 feet into stiff soil or bedrock as determined by the WTAI.
5. Class II permeable material given in Section 68-1.025, State of California Standard Specifications, July 1999 Edition.
6. Perforated pipe place perforations down, PVC pipe with a minimum diameter of four (4) inches, conforming to ASTM D-2751 SDR 35.



Note: Bottom of the subdrain trench and pipe should be sloped at least 1.0 percent.

WAYNE TING & ASSOCIATES, INC.	*Drainage Behind Retaining Wall*	*Figure No. 5*
	Scale: N/A	*Page No. 14*



Addendum Attachment E-4/p.15

WAYNE TING & ASSOCIATES, INC.	FOUNDATION DRAIN	Figure No. 6
GEOTECHNICAL CONSULTANTS	Scale: N/A	Page No. 15

APPENDIX B

Earthquake-induced Landslide Analysis

XSTABL File: DSTREET   9-16-05   8:49

```
****************************************
*              X S T A B L             *
*                                      *
*       Slope Stability Analysis       *
*              using the               *
*           Method of Slices           *
*                                      *
*       Copyright (C) 1992 - 2000      *
*  Interactive Software Designs, Inc.  *
*        Moscow, ID 83843, U.S.A.      *
*                                      *
*          All Rights Reserved         *
*                                      *
*  Ver. 5.204               96 - 1835  *
****************************************
```

Problem Description : dstreet

SEGMENT BOUNDARY COORDINATES
----

3 SURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	1.0	300.0	30.0	290.0	1
2	30.0	290.0	115.0	328.0	1
3	115.0	328.0	145.0	328.0	1

3 SUBSURFACE boundary segments

Segment No.	x-left (ft)	y-left (ft)	x-right (ft)	y-right (ft)	Soil Unit Below Segment
1	1.0	293.0	30.0	283.0	2
2	30.0	283.0	115.0	326.0	2
3	115.0	326.0	145.0	326.0	2

ISOTROPIC Soil Parameters

Addendum Attachment E-4/p.17

2 Soil unit(s) specified

Soil Unit No.	Unit Weight Moist (pcf)	Unit Weight Sat. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Parameter Ru	Pore Pressure Constant (psf)	Water Surface No.
1	110.0	120.0	540.0	16.00	.000	.0	1

2	130.0	130.0	1000.0	35.00	.000	.0	1

1 Water surface(s) have been specified

Unit weight of water = 62.40 (pcf)

Water Surface No. 1 specified by 2 coordinate points

****
PHREATIC SURFACE,
****

Point No.	x-water (ft)	y-water (ft)
1	30.00	282.00
2	115.00	320.00

A horizontal earthquake loading coefficient
of .270 has been assigned

A vertical earthquake loading coefficient
of .270 has been assigned

A critical failure surface searching method, using a random
technique for generating CIRCULAR surfaces has been specified.

100 trial surfaces will be generated and analyzed.

10 Surfaces initiate from each of 10 points equally spaced
along the ground surface between x = 30.0 ft
and x = 45.0 ft

Each surface terminates between x = 80.0 ft
and x = 120.0 ft

Unless further limitations were imposed, the minimum elevation
at which a surface extends is y = .0 ft

* * * * * DEFAULT SEGMENT LENGTH SELECTED BY XSTABL * * * * *

4.0 ft line segments define each trial failure surface.

ANGULAR RESTRICTIONS

The first segment of each failure surface will be inclined within the angular range defined by :

Lower angular limit := -45.0 degrees
Upper angular limit := (slope angle - 5.0) degrees

****
-- WARNING -- WARNING -- WARNING -- WARNING -- (# 48)
****
Negative effective stresses were calculated at the base of a slice. This warning is usually reported for cases where slices have low self weight and a relatively high "c" shear strength parameter. In such cases, this effect can only be eliminated by reducing the "c" value.
****

USER SELECTED option to maintain strength greater than zero

Factors of safety have been calculated by the :

* * * * * SIMPLIFIED BISHOP METHOD * * * * *

The most critical circular failure surface is specified by 29 coordinate points

Point No.	x-surf (ft)	y-surf (ft)
1	35.00	292.24
2	38.29	289.96
3	41.74	287.93
4	45.33	286.17
5	49.04	284.68
6	52.86	283.48
7	56.75	282.56
8	60.70	281.93
9	64.69	281.61
10	68.69	281.58
11	72.68	281.85
12	76.64	282.42
13	80.54	283.28
14	84.37	284.44
15	88.11	285.87
16	91.72	287.59
17	95.20	289.56
18	98.52	291.79
19	101.66	294.27
20	104.61	296.97
21	107.35	299.89
22	109.86	303.00

23	112.14	306.29
24	114.16	309.74
25	115.92	313.33
26	117.40	317.05
27	118.60	320.86
28	119.52	324.76
29	120.02	328.00

**** Simplified BISHOP FOS = 1.919 ****

The following is a summary of the TEN most critical surfaces

Problem Description : dstreet

	FOS (BISHOP)	Circle Center x-coord (ft)	Circle Center y-coord (ft)	Radius (ft)	Initial x-coord (ft)	Terminal x-coord (ft)	Resisting Moment (ft-lb)
1.	1.919	67.06	335.01	53.45	35.00	120.02	9.022E+06
2.	1.968	61.08	338.84	54.88	33.33	114.86	8.062E+06
3.	1.970	67.10	331.23	50.51	35.00	117.48	8.446E+06
4.	1.981	65.95	340.28	54.13	38.33	118.62	7.853E+06
5.	1.998	70.11	330.05	49.93	36.67	119.96	8.807E+06
6.	2.008	61.23	347.17	59.50	36.67	117.54	8.100E+06
7.	2.021	68.88	335.59	50.25	40.00	118.51	7.481E+06
8.	2.052	51.39	363.26	75.15	31.67	117.74	9.995E+06
9.	2.059	66.76	329.64	47.43	36.67	114.11	7.292E+06
10.	2.064	66.27	344.38	54.97	41.67	118.73	7.240E+06

* * * END OF FILE * * *

Probabilistic Seismic Hazards Mapping Ground Motion Page

User Selected Site

Longitude	-122.0567
Latitude	37.6769

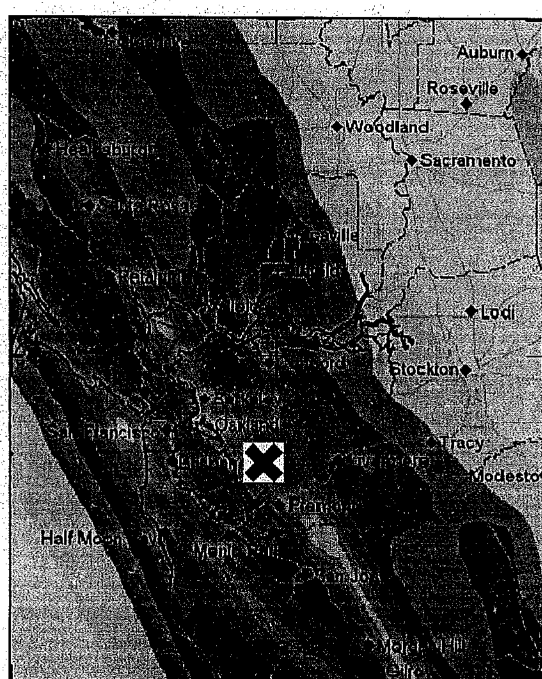
Ground Motions for User Selected Site

Ground motions (10% probability of being exceeded in 50 years) are expressed as a fraction of the acceleration due to gravity (g). Three values of ground motion are shown, peak ground acceleration (Pga), spectral acceleration(Sa) at short (0.2 second) and moderately long (1.0 second) periods. Ground motion values are also modified by the local site soil conditions. Each ground motion value is shown for 3 different site conditions: firm rock (conditions on the boundary between site categories B and C as defined by the building code), soft rock (site category C) and alluvium (site category D).

Ground Motion	Firm Rock	Soft Rock	Alluvium
Pga	0.712	0.712	0.712
Sa 0.2 sec	1.653	1.653	1.653
Sa 1.0 sec	0.633	0.716	0.826

NEHRP Soil Corrections were used to calculate Soft Rock and Alluvium.

Ground Motion values were interpolated from a grid (0.05 degree spacing) of calculated values. Interpolated ground motion may not equal values calculated for a specific site, therefore these values are not intended for design or analysis.



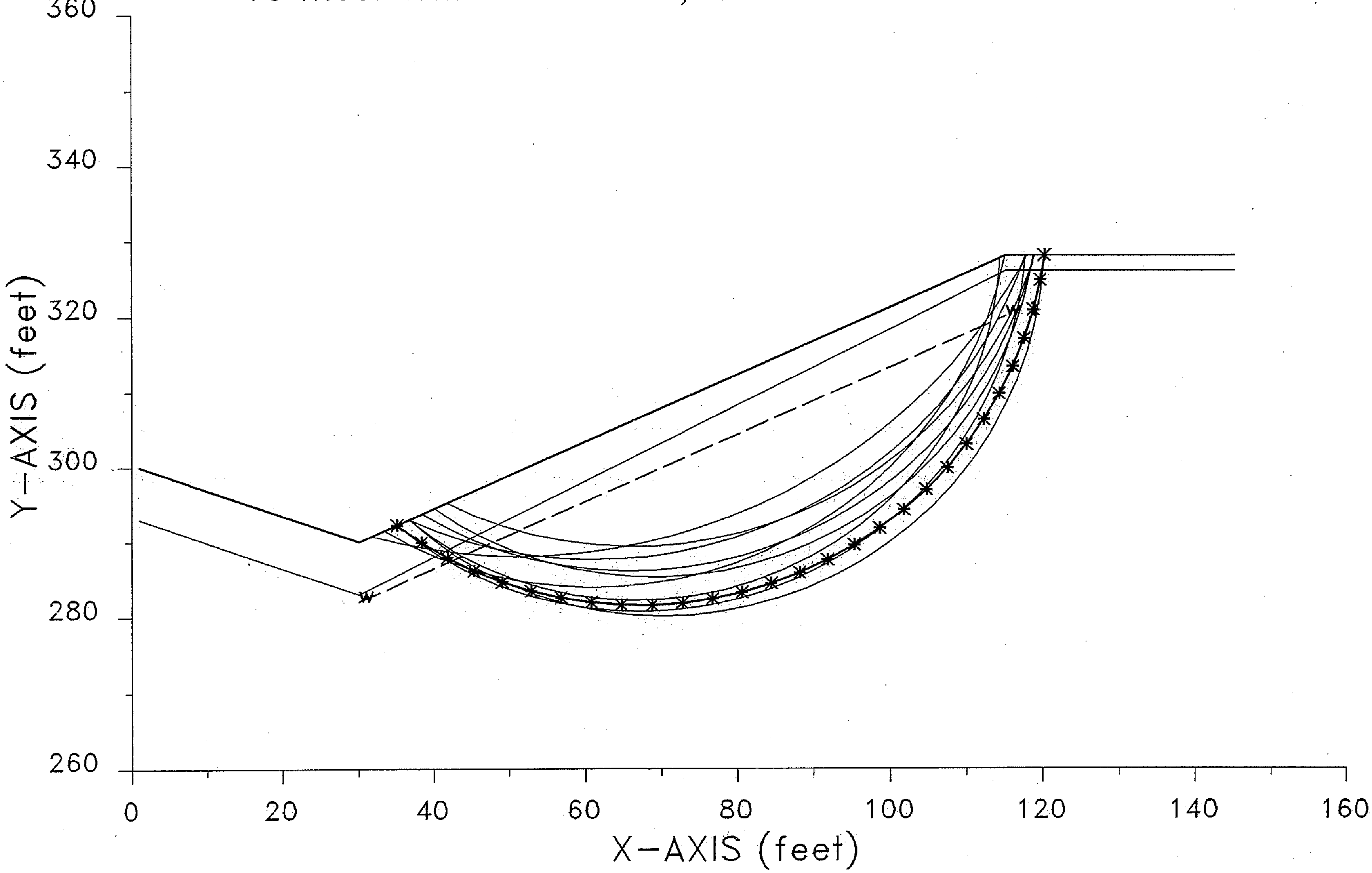
Addendum Attachment E-4/p.21

DSTREET 9-16-05 8:42

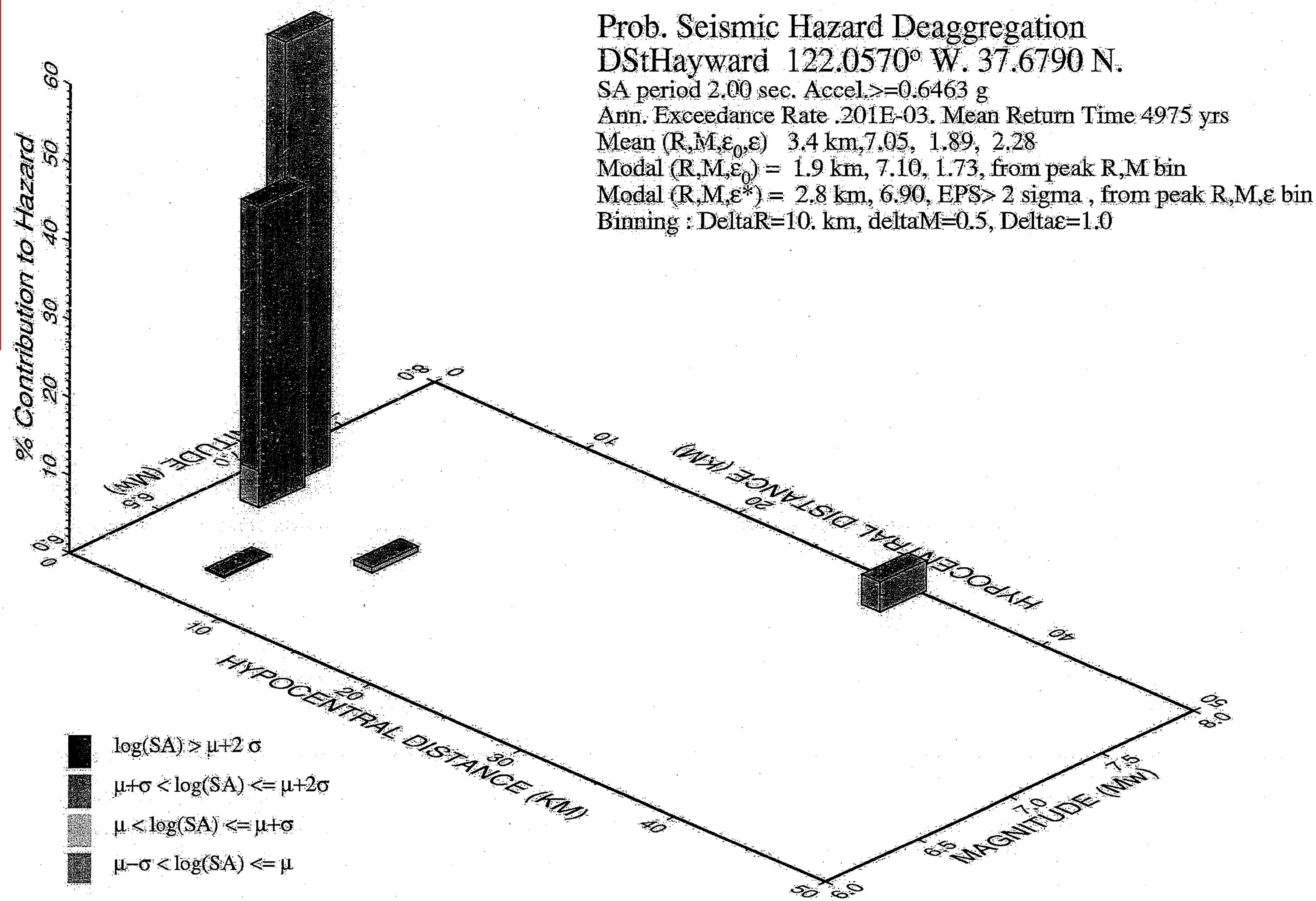
SECTION A - A

dstreet

10 most critical surfaces, MINIMUM BISHOP FOS = 1.919



Addendum Attachment E-4/p.23



GMT Sep 15 15:17 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs=760m/s top 30 m. USGS CGHT PSHA 1996 edition. Bins with lt 0.05% contrib. omitted

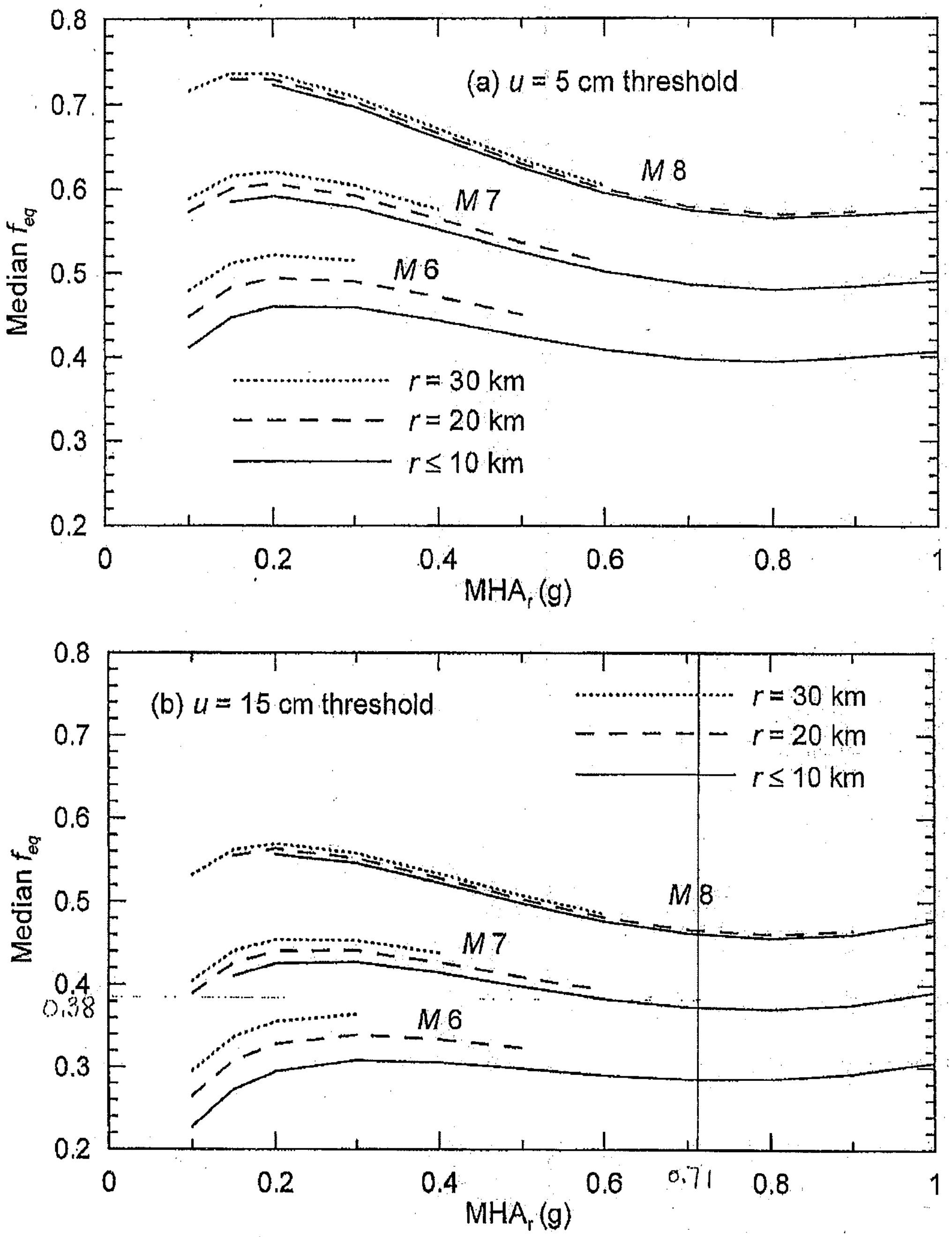
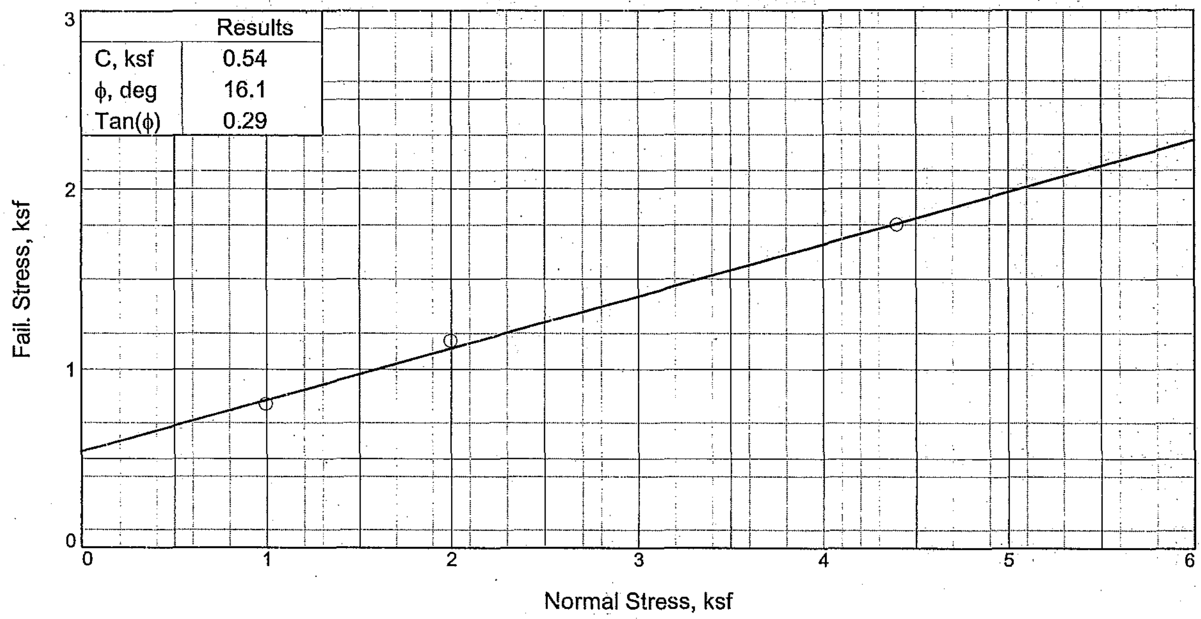
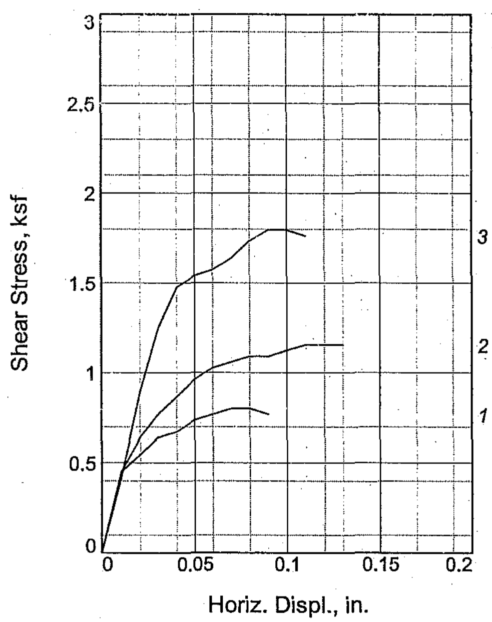


Figure 11.1. Required Values of f_{eq} as Function of MHA$_r$ and Seismological Condition for Threshold Displacements of (a) 5 cm and (b) 15 cm

$\bar{M} = 7.1$
$\bar{r} = 19$ km
$f_{eq} = 0.38$
$k_{eq} = 0.38 \times 0.71$
$= 0.27\, g$



Results	
C, ksf	0.54
ϕ, deg	16.1
Tan(ϕ)	0.29



Sample No.		1	2	3
Initial	Water Content, %	13.1	13.4	13.8
	Dry Density, pcf	94.9	95.4	96.0
	Saturation, %	45.8	47.0	49.3
	Void Ratio	0.7755	0.7675	0.7566
	Diameter, in.	2.39	2.39	2.39
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	24.4	23.8	25.0
	Dry Density, pcf	96.1	96.9	99.4
	Saturation, %	87.5	86.9	97.1
	Void Ratio	0.7533	0.7387	0.6965
	Diameter, in.	2.39	2.39	2.39
	Height, in.	0.99	0.98	0.97
Normal Stress, ksf		1.00	2.00	4.40
Fail. Stress, ksf		0.80	1.16	1.80
Displacement, in.		0.07	0.11	0.09
Ult. Stress, ksf				
Displacement, in.				
Strain rate, %/min.		1.00	1.00	1.00

Sample Type: Undisturbed
Description: Brown Sandy CLAY

Assumed Specific Gravity= 2.7
Remarks: *DS-CU* A fully undrained condition may not be attained in this test.

Figure ________

Client: Wayne Ting & Associates

Project: 2492 D - Street - #1855

Source of Sample: 1-1b **Depth:** 3'

Proj. No.: 245-015 **Date:** 9/2/05

DIRECT SHEAR TEST REPORT

COOPER TESTING LABORATORY

Tested By: MD **Checked By:** PJ